WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER
LOCAL PROTECTION

CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

DESIGN MEMORANDUM NO. 8

AUXILIARY CONDUIT SHAFTS

SITE GEOLOGY, FOUNDATIONS AND DETAILED DESIGN OF STRUCTURES

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.

SEPTEMBER 1976

44

407-756

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED-E

30 September 1976

SUBJECT: Park River Local Protection, Connecticut River Basin, Hartford, Connecticut, DM No. 8, Auxiliary Conduit Shafts, Site Geology, Foundations and Materials and Detailed Design of Structures

HQDA(DAEN-CWE-B)
WASH DC 20314

In accordance with ER 1110-2-1150, there is submitted for review and approval DM No. 8, Auxiliary Conduit Shafts, Site Geology, Foundations and Materials and Detailed Design of Structures for the Park River Local Protection Project.

FOR THE DIVISION ENGINEER:

Incl (10 cys)
as

JOHN Wm. LESLIE
Chief, Engineering Division

AMERICAN REVOLUTION BICENTENNIAL 1776-1976

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION
CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

DESIGN MEMORANDA INDEX

Number	Title	Anticipated Submission Date	Date Submitted	Date Approved
1	Hydrology		16 Feb 73	12 Apr 73
2	GDM - Phase I - Plan Formulation		30 Mar 73	16 Jul 73
2	GDM - Phase II - Project Design, Site Geology & Interior Drainage Part I - Box Conduit		30 Aug 74	18 Oct 74
2	GDM - Phase II - Project Design Part II - Auxiliary Conduit		24 Jan 75	3 Mar 75
3	Hydraulic Analysis		19 Feb 75	27 Mar 75
4	Concrete Materials Part I - Box Conduit		22 Apr 75	6 May 75
5	Embankment & Foundations Part I - Box Conduit		7 Feb 75	18 Mar 75
6	Pumping Stations		29 Nov 74	10 Jan 75
7	Detailed Design of Structures Part I - Box Conduit		25 Oct 74	4 Dec 74
8	Auxiliary Conduit Shafts Site Geology, Foundations & Detailed Design of Structures		30 Sep 76	
9	Auxiliary Conduit Tunnel Site Geology, Foundations, Concrete Materials & Detailed Design of Structures	Nov 76		

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WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION
CONNECTICUT RIVER BASIN
HARTFORD, CONNECTICUT

A. PERTINENT DATA

PURPOSE	Flood Control
LOCATION	
State	Connecticut
County	Hartford
City	Hartford
River	Park River & North and South Branch Park River
River Basin	Connecticut
PARK RIVER DRAINAGE AREAS	
Park River at the Mouth	78.7 Square Miles
North Branch Park River	27.7 Square Miles
South Branch Park River	47.0 Square Miles

RECORD OF MAJOR FLOODS

Year	Month	Peak Discharge, c.f.s.*
1936	March	5,400
1938	January	5,650
1938	September	5,320
1955	August	14,000
1955	October	6,420

*Gage at Riverside St. on Park River about 600 feet below junction of North and South Branches.

AREAS

Subject to flooding, Acres	3,000
Inundated 1955 flood of record, Acres	2,300
Properties protected	Industrial, commercial, residential and public

TWIN-RECTANGULAR BOX CONDUIT

Material — Reinforced Concrete

Conduit Section	Length in Feet	Half Section Width	Half Section Height
Existing (12,743 ft):			
Original (1944)	5,600	30'-0"	19'-4"
Section 1	1,213	34'-0"	26'-6"
Section 3	1,710	34'-0"	26'-6"
Section 6	1,460	36'-0"	27'-6"
Section 8	2,760	22'-0"	25'-0"
UNDER CONSTRUCTION (4,036 ft): (Scheduled completion date - 1980)			
Section 2	1,232	34'-0"	26'-6"
Section 4	1,337	34'-0"	26'-6"
Section 5	103	36'-0"	27'-6"
Section 7	1,044	22'-0"	25'-0"
Section 9	320	22'-0"	25'-0"

POPE PARK PUMPING STATION - (Formerly Riverside Pumping Station)

Location	Pope Park by Conduit Section 4
Capacity, CFS	75
Area Controlled, Acres	40
Pumps (No.)	3

ARMORY PUMPING STATION

Location	State Armory by Conduit Section 1
Capacity, cfs	120
Pumps (No.)	3

AUXILIARY CONDUIT

Length, feet	9,100
Size, Inside Dia.	22'-0"
Liner Material	Reinforced Concrete
Capacity	5,400 cfs
Predominant Excavation	Shale Rock

SYSTEM DESIGN CAPACITY (Conn. River Stage of 30 ft. MSL)

Park River Conduit	18,400 CFS
Auxiliary Conduit	5,400 CFS
Total	23,800 CFS

AUXILIARY CONDUIT SHAFTS

Intake Shaft

Inside Diameter	22'-0"
Depth, Total, Feet	143
Depth to Rock, Feet	64
Liner, Material and Thickness	Reinforced Concrete (15")

Outlet Shaft

Inside Diameter in Soil	30'-0"
Inside Diameter in Rock	22'-0"
Depth, Total, Feet	146
Depth to Rock, Feet	70
Liner in Soil, Material and Thickness	Reinforced Concrete (24")
Liner in Rock, Material and Thickness	Reinforced Concrete (15")
Height of Protection During Construction	Elevation 27.0 MSL

Local Drainage Shafts

Inside Diameter	4'-0"
Liner Material	Reinforced Concrete
Total Depth, Broad St. Shaft	175'
Depth the Rock, Broad St. Shaft	50'
Total Depth, Main St. Shaft	185'
Depth to Rock, Main St. Shaft	20'

B. INTRODUCTION

1. PURPOSE. The purpose of this memorandum is to present for review and approval the geotechnical and structural investigations and design criteria developed for the Auxiliary Conduit Shafts of the Park River Local Protection Project, Hartford, Connecticut, in accordance with ER 1110-2-1150. The basic criteria, loading assumptions, typical design computations, logs of explorations, definitive plans and other pertinent data are presented.

2. SCOPE. This memorandum incorporates the geological, soils and structural features associated with the design and construction of the shafts for the Auxiliary Conduit. The Auxiliary Conduit, a 9100 foot long 22 foot diameter tunnel through rock will be the subject of feature Design Memorandum DM-9, the final memorandum for the project. Duplication of data previously submitted and approved has been minimized. Design Memorandum No. 2, Phase II, Project Design, Part II-Auxiliary Conduit, approved 3 March 1975, contains the most recent project description. Scheme D, the deep rock tunnel scheme recommended in the above referenced memorandum, is now the adopted scheme and is the basis for this and future memoranda. Plate 8A-1 is a general plan of the project and identifies those features now under construction and those discussed in this memorandum. An updated estimate of the total project cost (Auxiliary Conduit and Conduit Extension) is inclosed.

3. STATUS OF PROJECT. Part I of the project, consisting of an inlet structure, five sections of twin box conduit, junction structure and two pumping stations, is currently under construction and scheduled for completion in January 1980. The junction structure combines the North and South Branches of the Park River and provides a connection to the intake shaft of the Auxiliary Conduit. Construction of the intake structure portion of the Auxiliary Conduit will be scheduled after completion of the junction structure.

4. DEVIATIONS FROM PREVIOUS MEMORANDA.

a. Intake Structure. The intake structure as now proposed differs in several respects from that presented in Design Memorandum No. 2, Phase II, Part II. The connection stub from the junction structure to the intake structure has been extended ten feet to incorporate the stop log feature and access opening. The stop log storage crypts and ventilation shaft remain part of the intake structure. The ventilation riser has been located over the center of the intake shaft. The transition from square to circular cross section has been relocated from the tunnel entrance to the top of the vertical shaft in rock. The radius of intersection between the shaft and tunnel has been reduced to provide a sufficient tangent length to include the transition section. In accordance with the second indorsement of Design Memorandum No. 2 the shaft will be driven to the tunnel invert and fill concrete used to form the radii.

b. <u>Outlet Structure</u>. The configuration and function of the outlet structure has changed from those set forth in Design Memorandum No. 2. The outlet shaft in addition to being a working shaft for tunnel construction will now also serve as a clean out and inspection shaft for the completed Auxiliary Conduit. The radius of curvature at the intersection between the shaft and tunnel has been reduced and a sump area provided in the tunnel at the bottom of shaft. The sump will provide a flat invert at the bottom of the shaft for landing equipment used in clean out operations. Steel plates curved to the intersection radius will cover the sump when the conduit is in use. The portion of the shaft in overburden has been increased in diameter to 30 feet to compensate hydraulically for the reduced radius at its intersection with the tunnel. It also provides a seat on competent rock outside the area to be excavated for the shaft in rock. The outlet shaft has been separated from the outlet box, located at the top of the shaft, by a waterstopped joint to allow for differential movement and to prevent longitudinal bending that may be induced in the shaft by varying loading conditions at the outlet box. Similar to the intake shaft the outlet shaft will be excavated to the tunnel invert and the intersection radii formed with fill concrete. Additional features in the outlet structure include a removable open grating platform on top of the outlet box for safety of the public.

c. <u>Clean Out and Inspection Shaft</u>. The clean out and inspection shaft proposed at station 37+00 of the Auxiliary Conduit in the project design memorandum has been deleted from the project. Studies indicate that the shaft is not required for economical tunnel construction and its inclusion would increase the cost of the project by approximately two million dollars. The primary function of the deleted shaft has been incorporated into the outlet structure as described in the preceeding paragraph. It is concluded that the advantages of having an additional access shaft in the completed tunnel do not warrant the expenditures required.

d. <u>Local Drainage Shafts</u>. The Metropolitan District of Hartford has plans to separate the storm drains and sanitary sewers, which are presently combined, in the area South of Park Street. It would be more economical for them to discharge the storm water runoff into the auxiliary conduit via shafts than to conduct it to pumping stations at the Park River conduit for discharge. The District has requested that shafts be provided at the intersection of Park and Broad Streets and at the location where the auxiliary conduit crosses Main Street. The two 4-foot diameter shafts will be constructed as part of this project but will be non-Federal costs. The locations of the proposed shafts are shown on Plate 8A-1.

C. HYDROLOGY AND HYDRAULICS

5. GENERAL. The hydrology for the project was presented in Design Memorandum No. 1. The hydraulic analysis and results of model studies made were presented in Design Memorandum No. 3. A determination has been made that the changes in configuration of the inlet and outlet shafts will not significantly alter flow characteristics in the Auxiliary Conduit.

D. CONCRETE

6. GENERAL. Concrete for the Auxiliary Conduit and shafts will be supplied from nearby commercial sources acceptable for use in civil works projects. Information on concrete was presented in Design Memorandum No. 4, Concrete Materials, Part I, Box Conduit. Any necessary updating or changes will be presented in Design Memorandum No. 9, Auxiliary Conduit Tunnel.

E. GEOTECHNICAL DESIGN

7. GEOLOGY. The project is located within the Connecticut Valley lowland, an elongated basin of sedimentary rocks approximately 20 miles in width extending from Long Island Sound northward through the center of the State of Connecticut. The bedrock of the basin is of Triassic age and is comprised of conglomerate, shale, and sandstone through which have intruded more recent sheets and dikes of basalt, a volcanic rock commonly called "trap." The basin is bordered on the east and west by faults which separate it from the New England Upland, an area of moderate relief comprised of maturely dissected resistant crystalline rocks. The relief of the basin is low except where faulting and differential weathering have left prominant ridges of resistant "trap" rock projecting above the valley floor. The general relief of the region presents a north-south trend which reflects the general strike of the trap rock ridges and of the sedimentary rocks which dip gently to the east. The bedrock of the region generally is blanketed by glacial till which mantles the bedrock surface, occurring at the surface in the highest parts of the lowland. In the lowland area, the till is buried beneath extensive lacustrine deposits of stratified sand and varved silt and clay. These deposits formed in glacial lakes generally having a spillway to the south which caused a southward dip in the varved silt and clay deposits of the major valleys. These varved fine grained deposits grade upward to silts and sands and become integral with the terraces of sands and gravels. The sand and gravel terraces formed in temporary lakes were controlled by the local spillways during glacial recession which in vicinity of the Park River in Hartford approximate elevation 45 M.S.L. Recent floodplain deposits of stream sediment and debris occur in local areas along the major streams. Depth and composition of these deposits is highly variable and dependant upon the area and stream gradient. The subsurface water level is controlled by the local stream gradient and topography with the upper clay layer creating an impervious boundary which controls subsurface discharge and produces local slope failures. Poor drainage in tributary streams has been caused by local damming of drainage systems during glacial recession compounded by trap rock ridges which have interrupted the east-west stream development.

8. SUBSURFACE INVESTIGATIONS.

a. Completed Investigations. Subsurface explorations were laid out and performed in conformance with current criteria and practice as described in Corps of Engineers manuals EM 1110-2-1801, "Geological Investigation," and EM 1110-2-1803 "Subsurface Investigation - Soil." Explorations for the shafts included seismic profiles and a drive sample boring at each location for the intake and outlet shafts. Information from an undisturbed sample boring near the intake shaft made for the box conduit (part 1) and from explorations made by the Corps of Engineers and other agencies near the outlet shaft were utilized in these investigations. Core borings in rock were performed with 4 x 5-1/2 and NX "M" Series bits and barrels. Boring FD 25 T was pressure tested and partially photographed with the borehole camera to obtain the orientation of the joint structure. Borehole photography and pressure testing will be conducted on FD 26 T and pump tests will be performed on both borings to determine probable rates of water infiltration in the shaft areas. Additional data will be included in DM No. 9 "Auxiliary Conduit Tunnel."

b. Laboratory Tests. All laboratory tests on soils, except as noted, were performed in accordance with the provision of EM 1110-2-1906, "Laboratory Soils Testing." All soil samples were visually classified according to the Unified Soil Classification System. The visual classifications of representative soil samples were confirmed by grain size analyses and Atterberg Limit tests. Selected soil samples were tested for natural moisture content, natural density, consolidation characteristics and shear strength. Rock cores were classified based on the system established in the report entitled "Engineering Classification of Rock Masses for the Design of Tunnel Support" by N. Barton, R. Lien and J. Lunde and pertinent portions of the draft EM 1110-2-2901 dated December 1973 entitled "Engineering and Design Tunnels and Shafts in Rock." A brief description of the laboratory procedures used in the rock testing program are included in Appendix A.

c. Presentations of Data. The results of the subsurface investigations are presented herein. The logs of the subsurface explorations and laboratory reports for tests performed on undisturbed samples from boring FD-13U are included in Appendix B. The detailed rock core log and typical test data for selected rock tests are included in Appendix A. Shear tests data for FD-13U are plotted on Plate No. B-1 in Appendix B.

d. Future Investigations. Borehole camera studies and rock pressure tests will be performed to obtain oriented joint structure and permeability of the rock at the intake boring FD-26T. For the sinking caisson procedure to be employed on the outlet shaft construction, a peripheral set of borings will be made to allow for the development of an accurate evaluation of the overburden conditions and the configuration of the sound rock surface.

9. CHARACTERISTICS OF FOUNDATION MATERIALS.

a. Distribution and Description.

(1) General. There are two types of foundation soil deposits present at both shaft locations. A glacial till deposit from 10 to 15 feet thick overlies the variable weathered bedrock surface. The

materials in this deposit consist of a compact gravelly silty sand (SM) with cobbles and boulders and occasional phases of gravelly sandy silt (ML) and silty sandy gravel (GM). Overlying the till is a zone of from 40 to 50 feet of soft varved clay consisting of alternating laminae of clay (CL and CH) and silt (ML). The laminae range in thickness from about an inch down to paper thinness. There are occasional seams of silty fine sand (SM).

(2) Intake Shaft. As described on the boring log for FD-13 the glacial till deposit is about 15 feet thick in the vicinity of this shaft and the varved clay is about 45 feet thick. Rock consists of a hard, reddish brown shale containing fractured zones with gray sandy to limy cross beds and local slickenside surfaces as depicted on Plate 8A-6. The subsurface water level is approximately 35 feet below the ground surface.

(3) Outlet Shaft. As indicated on the boring log for FD-25T, the glacial till deposit is about 12 feet thick at this shaft and the varved clay is about 42 feet thick. Overlying the varved clay there is a 28 foot zone of loose to moderately compact variable silty medium to fine sand (SM), gravelly silty sand (SM) and sandy silt (ML), the upper 15 feet of which consists of recent floodplain debris. The surface of this area has been filled to a depth of about 5 feet with stone riprap and gravel bedding, topsoil and gravelly silty sand (SM) mixed with brick fragments and cinders. The rock consists of a reddish brown thin-bedded to massive red shale with local sandstone zones. A cross section of the rock structure is shown on Plate 8A-6. The subsurface water level is at or slightly above, the level of the adjacent river.

b. Engineering Properties.

(1) Soils.

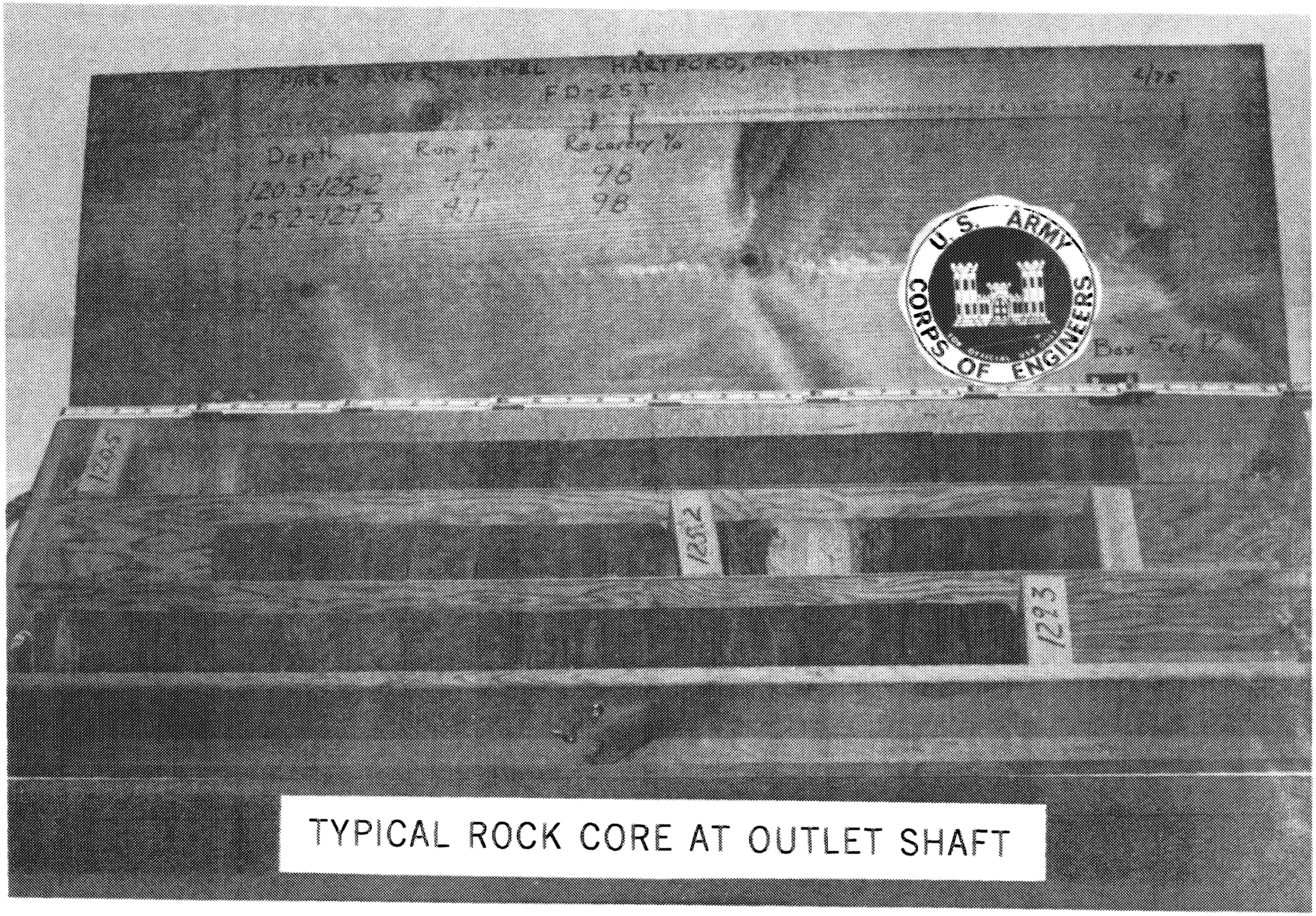
(a) Glacial Till. Materials in the glacial till deposit in the Hartford area are compact, well graded, gravelly silty sands, silty sandy gravels and gravelly sandy silts with fine contents between 25 and 55 percent. The materials range from nonplastic to slightly plastic with liquid limits of less than 25 and plasticity indices of less than 8. Natural water contents are between 8 and 15 percent. Specific gravity ranges from 2.69 to 2.75. Saturated natural unit weights are estimated to be between 140 and 150 pounds per cubic foot. For anticipated stress ranges these materials are estimated to have shear strengths equivalent to an angle of internal friction of 35 degrees based on past experience with similar soils. The estimated coefficient of subgrade reaction for these materials is on the order of 400 pounds per square inch per inch.

(b) Varved Clay. Reference is made to Design Memorandum No. 5, Embankments and Foundations, Part 1, Box Conduit, for a detailed evaluation of the engineering properties of the materials in the varved clay deposit. The material consists of alternating laminae of soft plastic clays and silts. Liquid limits of the clay bands range from 55 to 75 with plasticity indices between 25 and 45. The silts have liquid limits varying from 30 to 50 with plasticity indices between 0 and 20. Natural water contents range from 60 to 80 percent for the clays and from 35 to 50 percent for the silts. Specific gravities for both materials are between 2.75 and 2.85. Saturated natural unit weights are between 100 and 110 pounds per cubic foot. Consolidation test results for this and other projects indicated that the material has been preconsolidated to loads at least 0.5 TSF greater than present overburden stresses. For design purposes the undrained shear strength of the material is estimated to be 500 pounds per square foot.

(c) Silty Sands. The materials in the loose to moderately compact zone of silty sands at the outlet shaft are nonplastic silty medium to fine sand, gravelly silty sand and sandy silt. Saturated natural unit weights are estimated to be between 120 and 135 pounds per square foot. Shear strengths are estimated to be equivalent to an angle of internal friction of 30 degrees.

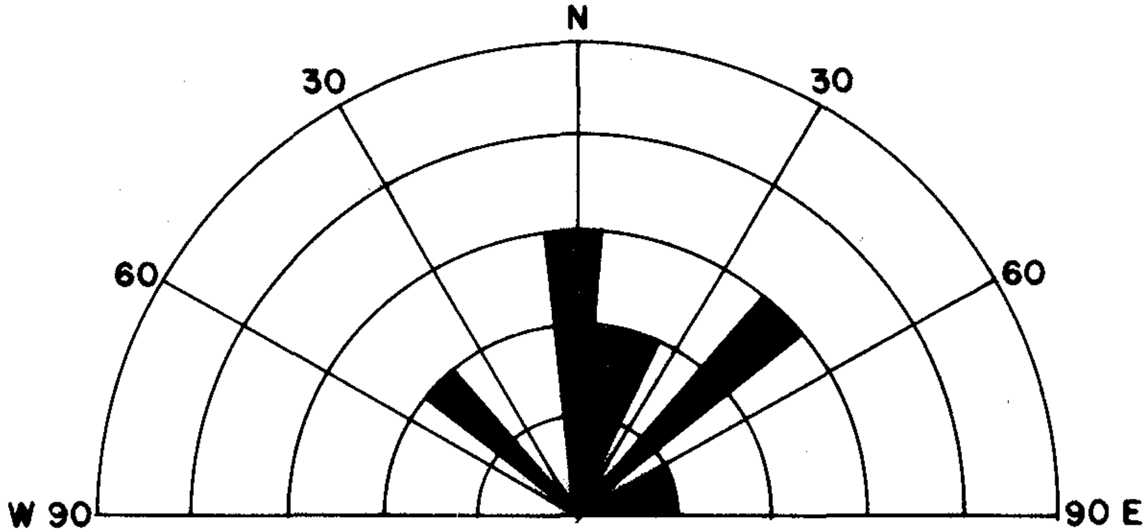
(2) Rock. The prevailing rock type in the area of the shaft structures is a red to gray shale having a variable bedding dip of 0^{o} to 30^{o}. A photograph of a typical section of rock core is shown on Figure 1, and a detail description of the rock structure at each shaft location is found in Appendix A. Borehole FD-25T is located at the outlet shaft and FD-26T is located at the intake shaft. Graphic representations of the rock joint structure at the shaft locations with the zone of pressure test losses at FD-25T are shown on Plate 8A-6. A joint structure orientation diagram is shown for FD-25T on Figure 2. Swell test conducted on a selected sample indicated relatively small maximum values of less than 0.4 of a percent swell in approximately 160 hours. A plot of the test results is shown in Appendix A, Figure A-5. The strength parameters and a summary of the rock conditions at each structure are as follows:

(a) Intake Structures. Detail examination of the rock core and analysis of the joint structure developed the following load factors with depth. Regional easterly dip of the bedding will cause the sloping rock structure related to bedding to be approximately 4.0' lower on the easterly or tunnel side of the shaft. The highly fractured and weathered rock zone between 54.2' and 64.2' has been considered as earth loading for the purpose of design. The following load factors have been developed for the rock structure at the centerline of the shaft.

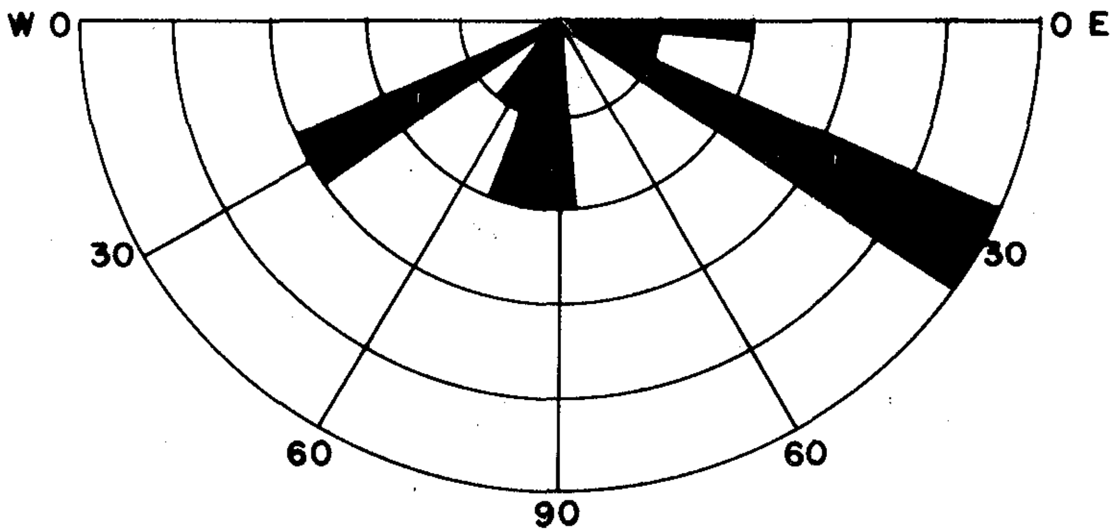


TYPICAL ROCK CORE AT OUTLET SHAFT

FIGURE 1



STRIKE



DIP

FD-25T
DIP & STRIKE ROSETTE

Depth Below Ground Surface	Horz. Load psf	Recommended Temporary Support
64.2 to 72.6	3,413	Rock Bolts 8.0'c.c.
72.6 to 107.0	1,138	Rock Bolts occ to 12.0'c.c.
107.0 to 116.5	4,550	Rock Bolts 6.0'c.c.
116.5 to 168.5	1,820	Rock Bolts 10.0'c.c.
168.5 to 177.5	3,413	Rock Bolts 8.0'c.c.
177.5 to 184.5	455	None to occasional
184.5 to 189.5	4,550	Rock Bolts 6.0'c.c.
189.5 to 198.0	2,275	Rock Bolts 9.0'c.c.

Bedding dip varies from 20° to 30° and with an average Ø of 27° and a C of .5 TSF measured along the natural joint surfaces, there would be only local areas of high angle dip where there would be a nonuniform loading on the shaft walls. High angle joints will govern the shape of the excavations. These joints which have tested at Ø of 32° and cohesive value of shearing of 96 TSF will have the capability of being restrained by effective rock bolting systems with a minimum of mine ties or other methods of internal support. For purposes of determining the support modulus an inplace horizontal modulus of elasticity for rock of 1×10^6 psi is assumed based on tests conducted on selected samples normal to the apparent bedding. Broken and fracture zones and high watertake sections, as noted on the boring log and Plate 8A-6, will require water control during construction. To provide an effective drainage system and provide immediate support to spalling zones, a reinforced shotcrete lining will be provided as a temporary water control and as safety measures against any loosened rock. Further evaluation will be made of the shaft excavation in rock following completion of the pressure testing and borehole photography.

b. <u>Outlet Shaft.</u> Examination of the rock core and borehole photography indicated that the rock is comprised of a thin-bedded to massive shale which is relatively sound with infrequent joints and a low angle of bedding dip. Regional dip will cause an average bedding displacement of approximately 2.0' across the shaft with the easterly wall or river side of the shaft being lower. The following load factors have been developed for the rock structure at the centerline of the shaft.

Depth Below Ground Surface	Horz. Load psf	Recommended Temporary Support
87.0 to 93.0	1,138	Bolts 10.0'c.c.
93.0 to 110.0	150	None to occasional
110.0 to 120.0	910	Bolts 12.0'c.c.
120.0 to 130.0	150	None to occasional
130.0 to 157.0	1,365	Bolts 10.0'c.c.
157.0 to 166.0	150	None to occasional
166.0 to 175.0	1,593	10.'c.c.
175.0 to 191.0	683	Occasional

Bedding dip varies from approximately 10^{o}-15^{o} and, with the average assumed Ø of 27^{o} and a cohesion value of shearing of 0.55 TSF, no nonuniform loading is expected as the result of displacement along the bedding. High angle joints will govern the shaping of the excavation. These joints which have tested at a cohesion value of shearing of 197 TSF and a Ø angle of 23^{o} can be effectively bolted. In view of the relative sound conditions of the rock and lack of broken and fractured zones rock bolts would be the only required temporary support. It is recommended that the shaft walls receive a minimal thickness (less than 6 inches) reinforced shotcrete lining to reduce spalling and to control water infiltration during construction. The horizontal modulus of elasticity is 3×10^{6} psi based on test data and the interpretation of the bedrock structures. This value is considerably higher here than at the intake structure due to the generally higher quality of the bedrock at the outlet shaft area.

10. SEISMICITY. The Hartford area is placed in the category of minor damage (zone 1) according to the seismic risk map developed by the Environmental Science Service Administration and the Coastal Geodetic Survey. In accordance with Engineering Technical letter No. 1110-2-109 dated 21 October 1970, hydraulic structures in Zone 1 will be designed to withstand minimum earthquake accelerations of .05 g. A single known fault zone is discussed in DM No. 2, Phase II Project Design, Site Geology and Interior Drainage, Paragraph 17.a. (1).

11. CONDITIONS OF SPECIAL ENGINEERING IMPORTANCE. The upper weathered zone of the rock surface is expected to be highly variable in thickness and to be comprised of clay and rock particles. This zone serves as an aquifer to the adjacent areas and will require special treatment to prevent excessive water losses during construction. The rock structure in the vicinity of the intake structure has been stressed and fractured by the adjacent basalt dikes. Special treatment of the shaft walls will be required during construction to control drainage and minimize rockfalls.

12. CONSTRUCTION CONSIDERATIONS. It is recommended that as the excavation of the shafts progress the rock be bolted and covered by a reinforced shotcrete lining. The rock bolts and reinforced shotcrete lining will stabilize the shaft walls during the period of tunnel construction. This temporary lining will be effective in reducing spalling and serve as an effective method of controlling seepage in the rock. Local zones may require limited use of mine ties or other means of intermediate supports to reduce spalling.

F. STRUCTURAL DESIGN CRITERIA

13. PURPOSE AND SCOPE. This section presents the design criteria, basic data, assumptions and procedures used in the structural design of the intake and outlet structures. Typical design computations are included in Appendix C.

14. DESIGN CRITERIA.

a. General. Allowable stresses, loading conditions, design assumptions and other criteria were based on applicable parts of the following references. Detailed criteria is also given for each of the major construction materials.

1. Working Stresses for Structural Design, EM 1110-1-2101, 1 November 1963.

2. Engineering and Design, Tunnels and Shafts in Rock, EM 1110-2-2901, Draft December 1973.

3. Standard Practice for Concrete, EM 1110-2000, 1 November 1971.

4. Details of Reinforcement-Hydraulic Structures, EM 1110-2-2103, 21 May 1971.

5. Conduits, Culverts and Pipes, EM 1110-2-2902, 3 March 1969.

6. Retaining Walls, EM 1110-2-2502, 29 May 1961.

b. Concrete. Concrete working stresses are in general accordance with ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63) as modified by EM 1110-1-2101, using concrete with a minimum ultimate compresive strength of 4000 pounds per square inch. Following is a listing of concrete working stresses used in design of the structures covered in this memorandum.

Flexure (extreme fiber in compression)	P.S.I. (max)
Shafts and intake box	1800
Outlet box	1400
Average Compressive stress	
Shafts	1000
Shear	
Beams-no web reinforcement	70

c. Steel Reinforcement.

(1) General. Details of reinforcement will be in accordance with EM 1110-2-2103, Details of Reinforcement-Hydraulic Structures.

(2) Grade and Working Stress. All reinforcement in the structures is designed for a working stress of 20,000 p.s.i. in tension. The reinforcement will be deformed bars made of new billet steel, intermediate grade (ASTM A-615, Grade 40), conforming to Federal Specification QQ-S-632C, Type II, Class B40.

(3) Spacing. The clear spacing between parallel bars will not be less than 1-1/2 times the nominal diameter of the bars except that in no case will the clear distance between parallel bars be less than 1 inch, or 1-1/2 times the maximum size of the coarse aggregate. Parallel reinforcement in two or more layers will have the bars directly over or beside each other.

(4) Minimum Cover for Reinforcement.

Item	Min. Cover (Inches)
Unformed Surface in Contact with Foundation	6
Weir at intake structure	6
All other surfaces	4

(5) Splices. All lap splices are in accordance with EM 1110-2-2103 and the ACI Building Code. Tension lap splices will be staggered longitudinally so that no more than half the bars are spliced at a section within the required lap length. Splices in main reinforcement at points of maximum moment will be avoided. The largest size of reinforcement bar required in the structures is No. 11. No butt splices in reinforcement are contemplated.

(6) Temperature and Shrinkage Reinforcement. Temperature and shrinkage reinforcement will be provided in slabs, walls and shaft linings where the main reinforcement extends in only one direction. This reinforcement will provide for a minimum ratio of steel to concrete area of .002 with one half located in each face.

(7) Minimum Flexural Reinforcement. Where reinforcement is required for flexure the amount provided will not be less than the temperature and shrinkage reinforcement in the opposite direction.

d. Structural Steel. The beams supporting the grating at the top of the outlet box are not considered to be laterally braced by the removable grating and have been sized accordingly assuming A 36 steel and a basic allowable stress of 18,000 p.s.i. The grating will be chosen based on allowable stress of 18,000 p.s.i. and a minimum material thickness of 1/4-inch. The supporting beams will be painted and the grating will be galvanized.

e. Increase in Normal Working Stress. The maximum allowable average compressive stress in shaft lining concrete is increased by one-third for the construction and maintenance conditions of loading assuming full external water head with no water inside the shaft. Allowable stresses will be increased by one-third for loading conditions with earthquake.

f. Waterstops. Rubber or polyvinylchlordie waterstops will be used in the joint between the outlet shaft and outlet box. Steel waterstops will be used in lift joints.

15. BASIC DATA AND ASSUMPTIONS.

a. Controlling Elevations.

Roof of Intake box	+40.40
Invert at start of intake box	+16.40
Invert of Auxiliary Conduit at C.L. of intake shaft*	-94.88
Top of intake shaft	-16.29
Invert of Outlet box	0.00
Top of walls at Outlet box	+18.00
Invert of Auxiliary Conduit at C.L. of outlet shaft*	-146.43
Top of stop logs at outlet box**	+12.00

*At intersection of extended tangents

**Consideration is being given to raising the top of stop log protection to Elevation 18.0. The computations prepared for this memorandum, however, assumes top of stop logs at Elevation 12.

b. Dead Loads.

Concrete		150 P.C.F.
Soils, Saturated		
	Till	150 P.C.F.
	Clay	110 P.C.F.
	Fill	120 P.C.F.
Rock		175 P.C.F.

c. Live Loads.

Water	62.5 P.C.F.
Grating and Framing at Outlet Box	100 P.S.F.

d. External Water Pressure. Hydrostatic pressure is assumed to act over the entire area in question under full available head. Specific assumption for various parts of the structures are given where the structures are discussed separately.

e. Internal Water Pressure. An internal bursting pressure equal to the difference in head between maximum hydraulic grade line and the roof soffit elevation of the intake structure was assumed in design of the intake structure. In design of the intake shaft internal pressure was computed for normal ground water conditions during pressure flow in the conduit.

f. Earth Pressure. Earth pressures were determined in general accordance with EM 1110-2-2502, retaining walls. Lateral pressure coefficients were assumed for either at rest or active pressure conditions to attain the critical loading for the various components of the structures. At rest coefficients of 0.43 and 1.00 were assumed for the till and clay soils respectively.

g. Rock Loads. Rock loadings as given previously in the section on Geotechnical Design were assumed in the design of the permanent shaft linings.

h. Earthquake. Earthquake forces will be considered in a loading case for the intake and outlet structures assuming five percent of the weight of all concrete and earth above the base slabs applied laterally. In addition, the difference between active and passive earth pressures on the walls will be applied opposite to the direction of the earthquake force.

i. Frost Protection. The foundations of all structures are located well below the depth required for frost protection in the area.

j. Uplift Factor of Safety. There is a minimum factor of safety of 1.28 against uplift of the outlet box under the maintenance condition with water outside the structure at elevation 12.0 (top of stop logs) and the structure dewatered. A factor of safety against uplift of 3.4 exists at the intake box assuming water on the outside to the top of the roof slab with no water inside.

k. <u>Location of Resultants</u>. Stability investigations show that the resultant of all loads fall within the middle third of the bases of both the intake and outlet structures under all conditions of loading.

l. <u>Sliding</u>. A minimum factor of safety against sliding of 2.5 will exist at the outlet structure with the shaft dewatered and water outside to elevation 12.0. Sliding is not a consideration in the stability of the intake structure.

G. PREPARATION OF DESIGN COMPUTATIONS

16. GENERAL. Two computer programs were utilized extensively in the analysis of structures presented in this memorandum. For analysis of the linings for the shafts the program "TUNNEL", as described in Technical Report C-73-2, "Computer Study of Steel Tunnel Supports" by U.S. Army Engineers Waterway Experiment Station, was employed. Through the facilities of McDonnell-Douglas Automation Co., the program "ICES-STRUDL-II" was used in the three dimensional analysis of the intake and outlet boxes. Representative printouts of computer results for each of these programs and computer graphics, generated by the "ICES-STRUDL-II" program, depicting the deflected intake and outlet boxes under load have been included in Appendix C.

17. INTAKE STRUCTURE.

a. General. The intake structure comprises the intake box and a vertical shaft in rock. The intake box receives flow from the Junction Structure and conducts it to the intake which leads to the Auxiliary Conduit below. The intake box will transition the flow from substantially horizontal to vertical through means of invert concrete curved to a 32 foot radius. The structure will measure approximately 60 feet by 30 feet in plan, will be 57 feet high and is to founded on sound bedrock. The intake shaft will be 22 feet in finished diameter and will be excavated in sound rock from the bottom of the intake box to the tunnel invert. A transition from square cross section to circular cross section is accomplished in the top twenty five feet of the shaft. Below the transition and a 10 foot tangent section, the centerline of shaft follows a vertical circular curve with a 33 foot radius until it joins the tunnel at a point of tangency fifty eight feet from the vertical centerline of shaft. The shaft will be excavated vertically to the tunnel invert and the curvature formed with fill concrete. The shaft will be lined with reinforced concrete. The amount of temporary support required will be minimal and will consist of rock bolts and wire mesh reinforced shotcrete. Two schemes were developed for construction of the intake box; a conventionally braced cofferdam scheme and a slurry wall cofferdam scheme, details of which are shown on Plates 8A-2 and 8A-3, respectively.

b. Construction Procedure.

(1) Conventionally Braced Cofferdam Scheme. This scheme requires installation of a steel sheet pile cofferdam around the proposed intake box leaving sufficient space for wales and forming. As excavation proceeds the cofferdam would be braced internally. The shaft would be excavated and material removed through the cofferdam. Upon completion of the shaft the intake structure would be constructed and the cofferdam removed.

(2) Slurry Wall Cofferdam Scheme. This scheme consists of constructing a vertically reinforced concrete cofferdam by the slurry trench method. The resulting cofferdam would have 32" thick walls. Temporary bracing would be installed internally while the excavation is continued to the top of the shaft. After completion of the shaft a series of five horizontal concrete frames would be constructed up to the top of the weir at El. 16.4. Construction of the remaining walls and roof would complete the structure.

(3) Comparison of Schemes. The slurry wall scheme would provide a more watertight cofferdam, would require fewer levels of bracing and less excavation than the conventionally braced cofferdam scheme. The conventionally braced cofferdam is estimated to cost less than the slurry wall scheme and its cost is included in the project estimate. Both schemes are feasible however and will be presented as alternates on the contract drawings.

c. Loading Conditions.

(1) Intake Box. The loading cases considered in the design of the intake box are as follows:

Case 1. Maintenance Condition - Full dead load, at rest lateral earth pressure, ground water at elevation 20 with no water inside the box.

Case 2. Operating Condition - Full dead load, at rest lateral earth pressure, no ground water and an internal pressure equivalent to water at elevation 46.7, the maximum hydraulic grade at the intake.

Case 3. Stability Condition - Full dead load at rest lateral earth pressure, ground water outside to elevation 34 with no water inside.

Loading cases 1 and 2 are graphically presented on Plate 8A-2. Additional loadings including earthquake and active lateral earth pressures will be investigated during the preparation of contract documents.

(2) Shaft Lining. The loading cases considered in the design of the shaft lining are as follows:

Case 1. Full external water head, including internal water pressure.

Case 2. Full rock load, no water.

Case 3. Full external water and rock load combined including internal water pressure.

Case 4. Grout pressure only.

Case 5. Construction condition; full external water head only with allowable stresses increased by one third.

Due to gravity placement of concrete and the small internal pressure to be experienced by the shaft it is felt that pressure grouting between the liner and rock will not be required.

The shaft was checked for condition 1 assuming full bursting pressure, equivalent to water at elevation 46.7, with external groundwater at elevation 20 resulting in a net internal pressure of 11.6 psi. The bottom of the shaft was investigated for loading condition 5 assuming groundwater at elevation 20 with the shaft dewatered resulting in a maximum uniform external pressure of 50 psi. Rock loading for cases 2 and 3 were assumed as given previously.

d. Analysis.

(1) Intake Box. In simulating a computer model for analysis of the intake box certain assumptions were made. All wall to wall, roof to wall and wall to weir connections were considered to be continuous. The walls and frames were assumed to be fixed at the weir. The structure was assumed pinned at the base against vertical motion. Due to the construction technique, the slurry trench walls were considered to span vertically only, with the five concrete frames, spanning horizontally. The slurry wall acts to distribute load to the frames and serves as the primary structural member from the top horizontal frame to the roof slab. The walls in the conventionally braced cofferdam scheme span in both directions and are reinforced accordingly. A representative sample of the input, output and computer graphics of deflected shapes are included in Appendix C.

(2) Shaft Lining. The shaft lining was analyzed as a thin shell subjected to uniform internal and external pressures and lateral rock loads. Under loading cases 2 and 3 it was assumed that the rock interacts with the lining. A rock modulus of 1,000,000 pounds per cubic inch was assumed in the ring analysis when considering rock loading. An additional criterion was applied requiring a minimum ratio of area of transverse reinforcement to area of concrete of .0025. It was this criterion which determined the amount of transverse reinforcement specified for the circular shaft. To allow for required cover of reinforcement and proper placement of the concrete, a lining thickness of 15 inches was selected. The square and transition sections of the shaft will be more heavily reinforced.

18. OUTLET STRUCTURE.

a. General. The outlet structure comprises the outlet box concrete apron and a vertical circular shaft partly through earth and partly through rock. The outlet box will direct the discharge from the shaft to the river while preventing erosion of the surrounding soil. Stop logging features of the box will provide for dewatering of the shaft and tunnel during maintenance and inspection operations. The shaft will have an inside finished diameter of 30 feet from the invert of the outlet box to elevation of sound rock, a depth of approximately 70 feet. The shaft will transition from 30 feet to 22 feet at the top of the rock and continue through the rock with a finished inside diameter of 22 feet to the tunnel invert, a depth of approximately 77 feet. The outlet box will measure 66 feet by 95 feet in plan inside and will be founded on existing granular fill material some 15 feet below existing ground level. The remaining granular soil mantle overlying the clay will be 15 feet thick under the invert slab. To allow for differing foundation conditions between the shaft and outlet box, a waterstopped joint will be provided at their intersection. To provide protection for the shaft and tunnel against flooding during construction an earthen berm will be constructed to elevation 25.0 and the shaft constructed through it. The top of the shaft will be maintained a minimum of 2 feet above berm elevation until completion of the tunnel, thereby providing protection against the theoretical 40 year storm. The berm will be large enough to provide adequate work area for construction operations and those portions not required in final grading of the project will be removed after completion of the outlet structure.

b. Construction Procedure.

(1) Shaft in Soil. It has been concluded that the most feasible and least costly method of constructing the shaft in soil is to cast the shaft above the earth berm in lifts and sink it open ended to the

rock surface below. The shaft will be sunk by excavating in the wet with jetting outside as necessary to reduce frictional resistance between the shaft and soil. The surface of the concrete will also be treated to further reduce friction. The bottom of the shaft will be contoured to match the shape of the rock surface to minimize the problem of sealing the interface of the shaft and rock surface during dewatering. The total height of shaft to be constructed in this manner, including the temporary portion, is 95 feet. It is felt that pneumatic methods will not be required during construction of the shaft. Problems that are encountered in excavating the till mantle or in seating the shaft on rock can be observed by divers and appropriate action taken. Grouting of the rock and area immediately outside the bottom of the shaft will aid in sealing the flow of water and can be accomplished from the surface.

(2) Shaft in Rock. Once the shaft in earth is constructed, properly seated on rock and the flow of water controlled, excavation for the shaft in rock can begin. The top 20 feet of the shaft in rock will require light excavating methods so that the shaft in soil is not disturbed. Bolting and shotcreting will closely follow the excavation to minimize water control and dangers of falling rock.

(3) Outlet Box. Upon completion of the shaft and tunnel, a braced sheet pile cofferdam will be constructed within the earthen berm for construction of the outlet box in the dry. The cofferdam sheeting will be driven deep enough to provide sufficient cutoff so that normal pumping will keep the water below the bottom of excavation during construction. After completion of excavation and dewatering operations the temporary shaft above elevation -3.0 will be removed and the outlet box constructed. After completion of the outlet box the earthen berm will be removed to the level of final grading, the cofferdam removed and final riprap protection placed.

c. Loading Conditions.

(1) Outlet Box. The three conditions of loading investigaed in the design of the outlet box are as follows:

Case 1. Construction Condition - Dead load of the concrete structure only in the dewatered cofferdam.

Case 2. Operating Condition - Full dead load, at rest lateral earth pressure and river level at the bottom of the invert slab.

Case 3. Maintenance Condition - Full dead load, at rest lateral earth pressure, no water inside and water outside the box to the top of the stop logs at elevation 12.0.

The loading cases are presented graphically on Plate 8A-5. Additional loading cases including earthquake and construction surcharges with a one third increase in allowable stress will be investigated during preparation of contract documents. The U shaped concrete apron, which is a separate monolith, lies outside the dewatered area thereby eliminating loading condition 3 above from consideration in design of that section.

(2) Shaft in Soil. The following two loading conditions were investigated in design of the portion of the outlet shaft in soil.

Case 1. Construction Condition - The shaft constructed to elevation 27.0, the temporary berm at elevation 25.0, a construction equipment surcharge loading of 500 pounds per square foot at the top of berm, the river level at elevation 27.0 and no water in the shaft.

Case 2. Maintenance Condition - Construction completed, shaft dewatered and river level below the invert slab.

It is assumed that the internal pressure will never exceed the external pressure on the outlet shaft in soil. Several cross sections on the shaft were investigated for the loading case above to find the critical combination of thrust coincident with bending caused by unbalanced lateral earth pressures. In addition to investigation of the cross section the shaft was also investigated as a cantilevered column, fixed at the top of rock, subjected to unbalanced lateral earth loading.

(3) Shaft Lining in Rock. The loading conditions for design of the shaft lining in rock at the outlet structure are the same as those presented for the intake structure. Loading condition 5 results in a maximum external uniform pressure of 75 psi assuming the river at elevation 25.0 with the shaft dewatered.

d. Analysis

(1) Outlet Box. The outlet box was analyzed for stability and structural adequacy. The stability analysis indicated that the invert slab must be extended beyond the periphery walls to meet stability criteria under loading case 3. A computer model was developed for a three dimensional analysis of the box assuming an elastic foundation and three foot thick slab and walls. This assumption proved to be reasonable and the reinforcing for the structure was designed accordingly. Typical examples of computer input, output and graphical presentations of deflected structure shapes are included in Appendix C.

(2) Shaft in Soil. The concrete shaft in soil was analyzed as a thin shell subjected to uniform external radial pressure and lateral earth pressure applied similar to loading on the roof of a tunnel. The concrete shell was designed as a beam with combined bending and axial compression. The bending produced by lateral earth pressures from construction surcharges can be applied in any direction and produce tension stress on both the inner and outer faces of the shell. This requires complete rings of transverse reinforcement at inner and outer surfaces of the shaft concrete. A shell thickness of 24" was selected though it is somewhat more than required by design. The added thickness will reduce the amount of ballast and jetting required to sink the shell and reduce the bearing pressure on the supporting rock.

(3) Shaft in Rock. The analysis of the portion of the outlet shaft in rock is the same as that described for the intake shaft.

19. LOCAL DRAINAGE SHAFTS. The drainage shaft at Broad Street will be constructed through 100 feet of bedrock and 50 feet of granular overburden. At the Main Street location twenty feet of soil overlies the 150 of rock to be bored. It is expected that the shafts in rock will be bored from the roof of the tunnel up to the top of rock. The shafts in soil will be constructed of precast concrete pipe sections set into augered holes. The need for lining these shafts will be determined on the basis of rock jointing and ground-water conditions during tunnel design. Requirements for the structures at the top of the shafts will be determined by the Metropolitan District.

H. CONSTRUCTION SCHEDULE

20. CONSTRUCTION SCHEDULE.

a. General. The Box Conduit feature and the Auxiliary Conduit feature are sufficiently different and independent such that construction can be initiated independently. Construction of the Box Conduit was initiated in July 1976 and will run for a period of 4 construction seasons (3-1/2 years). The Auxiliary Conduit would commence approximately 1 year later than the box conduit construction start and would run for a period of 3 years. The schedule as set forth herein is considered reasonable and one that takes into account economics.

b. Auxiliary Conduit. It is assumed that the contract will be awarded in early spring of 1977, the 2nd construction season. The phases of construction are briefly outlined below, whereas the details of construction were more fully discussed in Section H, "Construction Procedure and Diversion Plan" of GDM No. 2, Phase II, Part II - Auxiliary Conduit.

(1) Second Construction Season (First Year Auxiliary Conduit Construction. The Contractor is expected to concentrate on the construction of the river shaft. It will take the Contractor approximately 6 months to prepare the site, construct and lower the caisson into place, grout-seal the interface between caisson and bedrock and complete the mucking out of the caisson. An additional 2 months will be required to continue the vertical shaft through rock down to EL-147 msl and an additional month will be required to set up his horizontal tunneling equipment. The Contractor will then proceed with his tunneling operation from the river shaft driving upward to the Pope Park end. The tunneling operation is expected to proceed uninterrupted throughout the winter. The tunneling heading should advance at a rate of approximately 750 ft./month based upon round-the-clock operations, 6 days a week. As the rock excavation progresses, the rock anchors are to be installed. By the end of the 12th month (3rd month of tunneling) the Contractor is expected to be at Sta. 30+00.

(2) Third Construction Season (Second Year Auxiliary Conduit Construction).

(a) Tunneling. The Contractor is expected to continue tunneling to the intake shaft at Pope Park. By the end of the second year the tunneling operation is expected to be complete.

(b) River Shaft. Once the river shaft is no longer needed for rock removal the Contractor is expected to proceed to modify the river shaft. At the junction between the tunnel and the river shaft, rock will have to be carefully removed to provide the necessary curvature for the bend; this is to be followed by the installation of the concrete lining both in the tunnel and in the shaft. At the same time the Contractor is expected to construct the single-wall braced steel sheet cofferdam around the outfall structure (top of river shaft).

(c) Intake Shaft. Towards the middle of the 2nd construction season, the Contractor is expected to initiate construction at the intake structure. He will initiate the installation of the earth support system. During the winter months he will move the overburden and excavate the necessary rock down to the tunnel invert and initiate the installation of the concrete liner.

(3) 4th Construction Season (Third Year Auxiliary Conduit Construction).

(a) Tunnel. Installation of the concrete liner is to be completed.

(b) Intake Shaft. The Contractor will form and place the concrete, backfill, remove the cofferdam if required and perform final grading, landscaping and cleanup.

(c) River Shaft. The Contractor is expected to excavate to grade, remove the segment of the caisson above El. -3.0 msl, form and place the necessary concrete, remove the cofferdam and excavate and place the rock paving. The necessary grading and cleanup would complete the river phase of the project.

I. COST ESTIMATE

21. Conduit Extension. The contract for the first half of the local protection project was advertised and the contract awarded to Vicon Construction Company of New Jersey for the bid price of $23,300,000. The bid price was approximately $5,282,000 less than the Government estimate of $28,582,155.

22. Auxiliary Conduit. The previous detailed cost estimate of $38,390,000 for the Auxiliary Conduit was included in Design Memorandum No. 2, Phase II, Part II, Auxiliary Conduit dated 24 January 1975. The estimate was based upon a price level of November 1974. For comparison purposes, the estimate was updated to the August 1976 price level by increasing the cost by 17% (per ENR). The cost, updated, would be $44,900,000.

The effects on project cost due to the more advance design of the shafts, (primarily the deletion of the clean out shaft) results in a overall reduction in cost of $1.9 million. The present estimated total cost of the auxiliary conduit portion of the project is $43,000,000.

The design of the tunnel will have a major bearing on the cost of the project. Upon completion of the tunnel design, a new detailed cost estimate will be prepared and included in DM No. 9.

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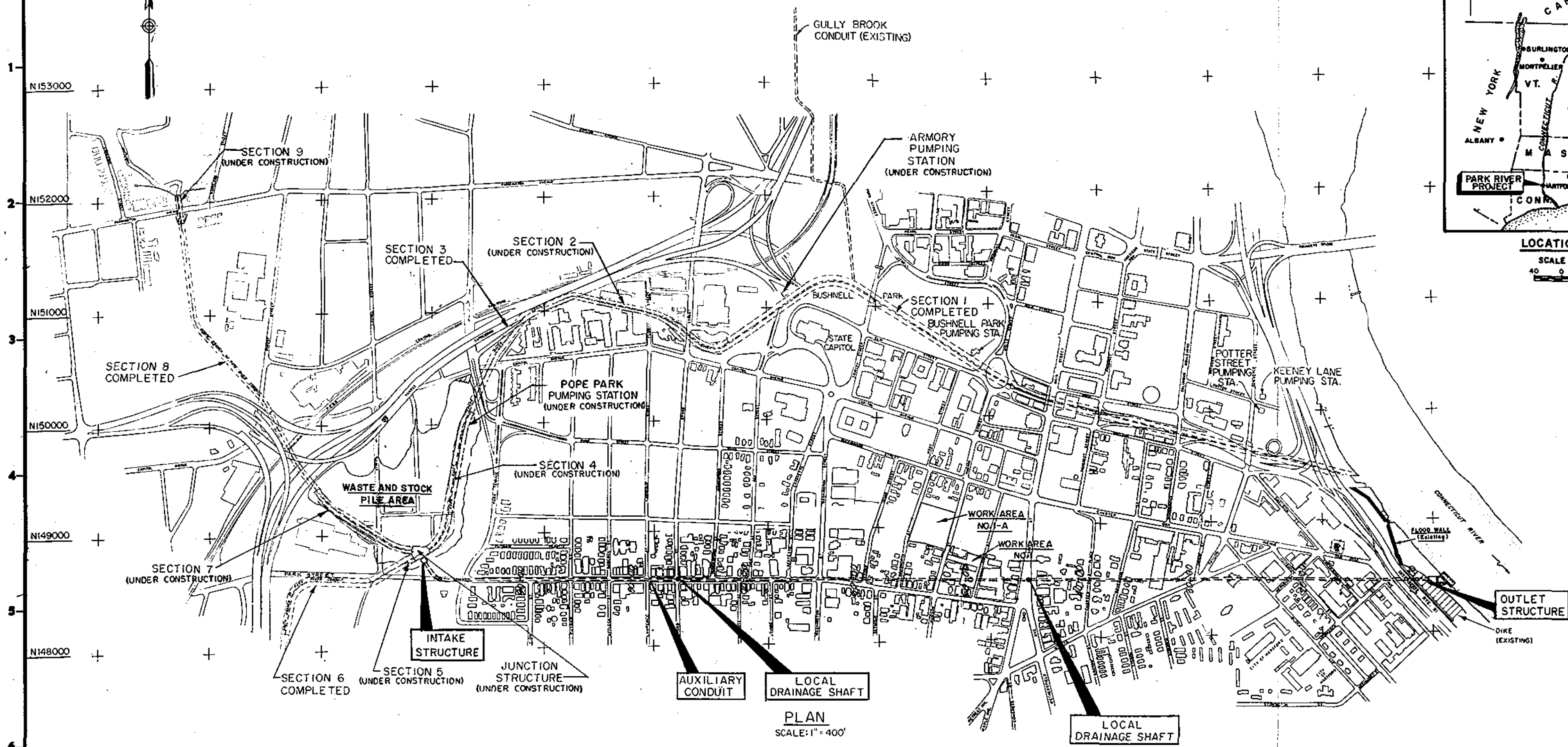
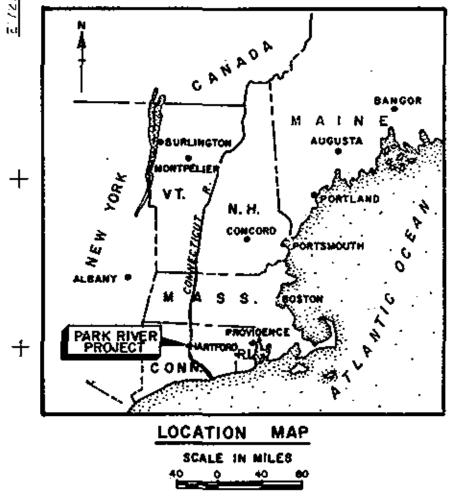


PLAN

SCALE: 1" = 400'



LOCATION MAP

SCALE IN MILES

40 0 40 80

GENERAL NOTES:

1. Elevations refer to Mean Sea Level Datum.
2. All elevations are plus except as noted otherwise.
3. Plane coordinates refer to Connecticut Lambert Grid System.
4. For Plans and Profiles of Auxiliary Conduit see D.M. No 2 Phase II, Project Design Part II, Auxiliary Conduit.

SAFETY PAYS

GRAPHIC SCALES

1" = 400' 400' 0 400' 800'

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT

PARK RIVER LOCAL PROTECTION

GENERAL PLAN

HARTFORD CONNECTICUT

APPROVED DATE

SCALE SPEC. NO.

DRAWING NUMBER

SHEET

PLATE 8A-1

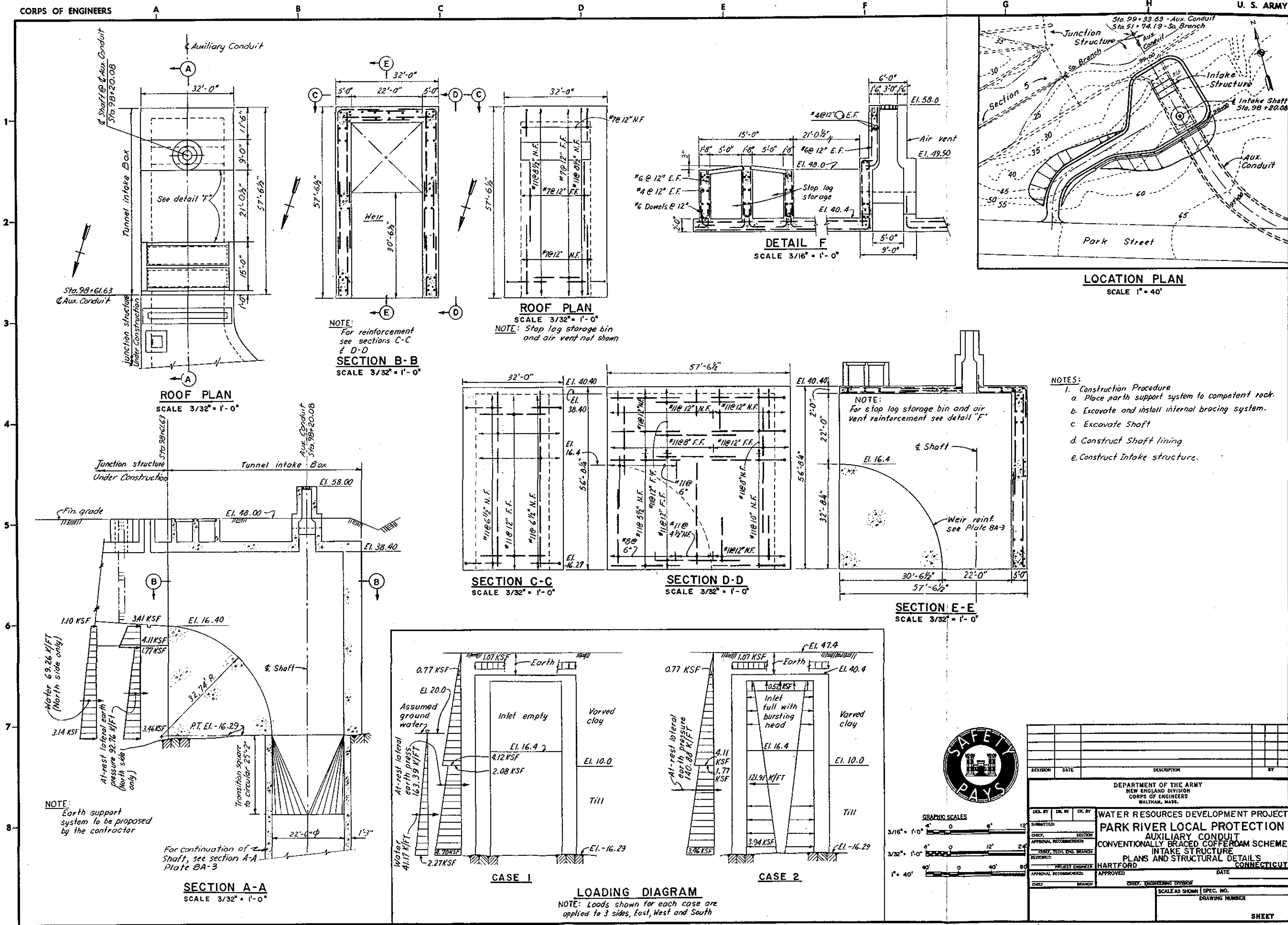
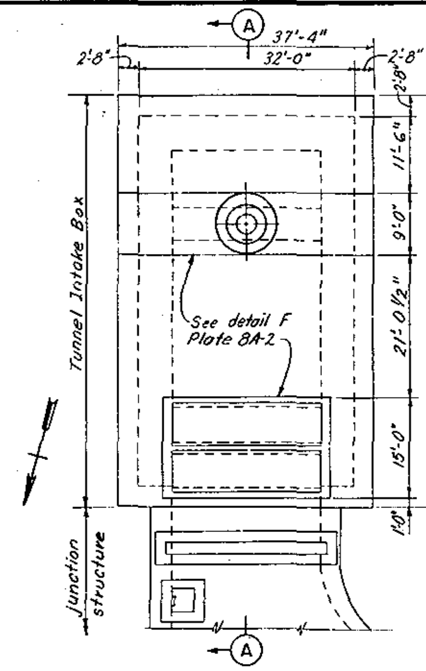
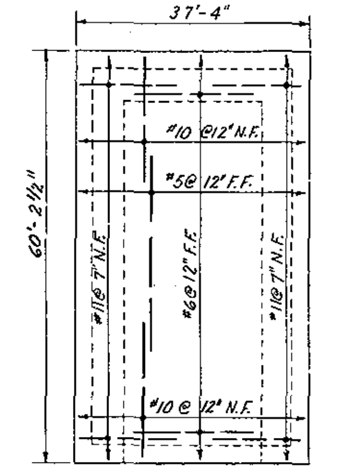

CORPS OF ENGINEERS
U. S. ARMY
ROOF PLAN
SCALE 3/32" = 1'-0"
¢ Auxiliary Conduit
¢ Shaft @ ¢ Aux. Conduit Sta. 98+20.08
Tunnel intake Box
See detail "F"
Sta. 98+61.63 ¢ Aux. Conduit
junction structure Under Construction
SECTION B-B
SCALE 3/32" = 1'-0"
Weir
NOTE: For reinforcement see sections C-C & D-D
ROOF PLAN
SCALE 3/32" = 1'-0"
NOTE: Stop log storage bin and air vent not shown
DETAIL F
SCALE 3/16" = 1'-0"
Air vent
Stop log storage
El. 58.0
El. 49.50
El. 48.0
El. 40.4
LOCATION PLAN
SCALE 1" = 40'
Junction Structure
Sta. 99+33.63 - Aux. Conduit
Sta. 51+74.19 - So. Branch
Intake Structure
¢ Intake Shaft Sta. 98+20.08
Section 5
Aux. Conduit
Park Street
SECTION C-C
SCALE 3/32" = 1'-0"
SECTION D-D
SCALE 3/32" = 1'-0"
SECTION E-E
SCALE 3/32" = 1'-0"
NOTE: For stop log storage bin and air vent reinforcement see detail "F"
¢ Shaft
Weir reinf. see Plate 8A-3
NOTES:
1. Construction Procedure
a. Place earth support system to competent rock.
b. Excavate and install internal bracing system.
c. Excavate Shaft
d. Construct Shaft lining
e. Construct Intake structure.
SECTION A-A
SCALE 3/32" = 1'-0"
Junction structure Under Construction
Tunnel intake Box
Fin. grade
El. 58.00
El. 48.00
El. 38.40
El. 16.40
P.T. El. -16.29
32.74' R.
¢ Shaft
Transition square to circular 25'-2"
Water 69.26 K/FT (North side only)
At-rest lateral earth pressure 92.76 K/FT (North side only)
NOTE: Earth support system to be proposed by the contractor
For continuation of Shaft, see section A-A Plate 8A-3
LOADING DIAGRAM
NOTE: Loads shown for each case are applied to 3 sides, East, West and South
CASE 1
CASE 2
Inlet empty
Inlet full with bursting head
Assumed ground water
Varved clay
Till
Earth
SAFETY PAYS
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
CONVENTIONALLY BRACED COFFERDAM SCHEME
INTAKE STRUCTURE
PLANS AND STRUCTURAL DETAILS
HARTFORD CONNECTICUT
SCALE AS SHOWN
DRAWING NUMBER
SHEET

PLATE 8A-2

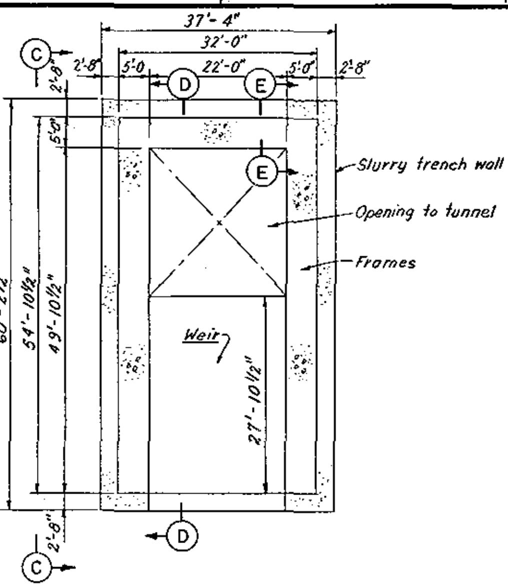
CORPS OF ENGINEERS — U. S. ARMY



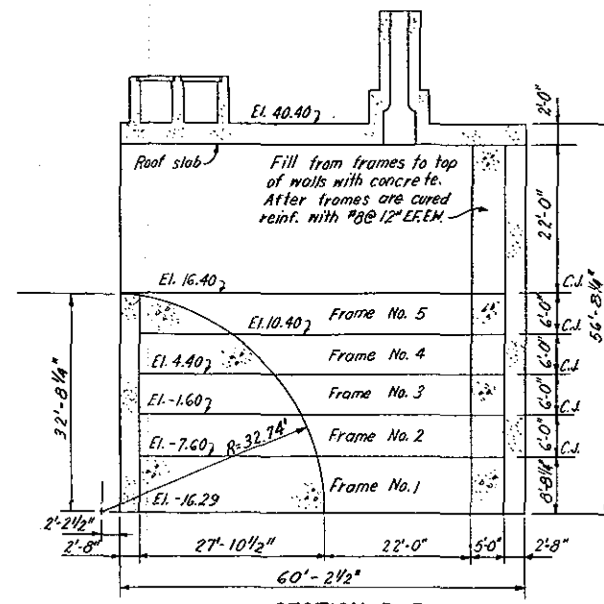
PLAN



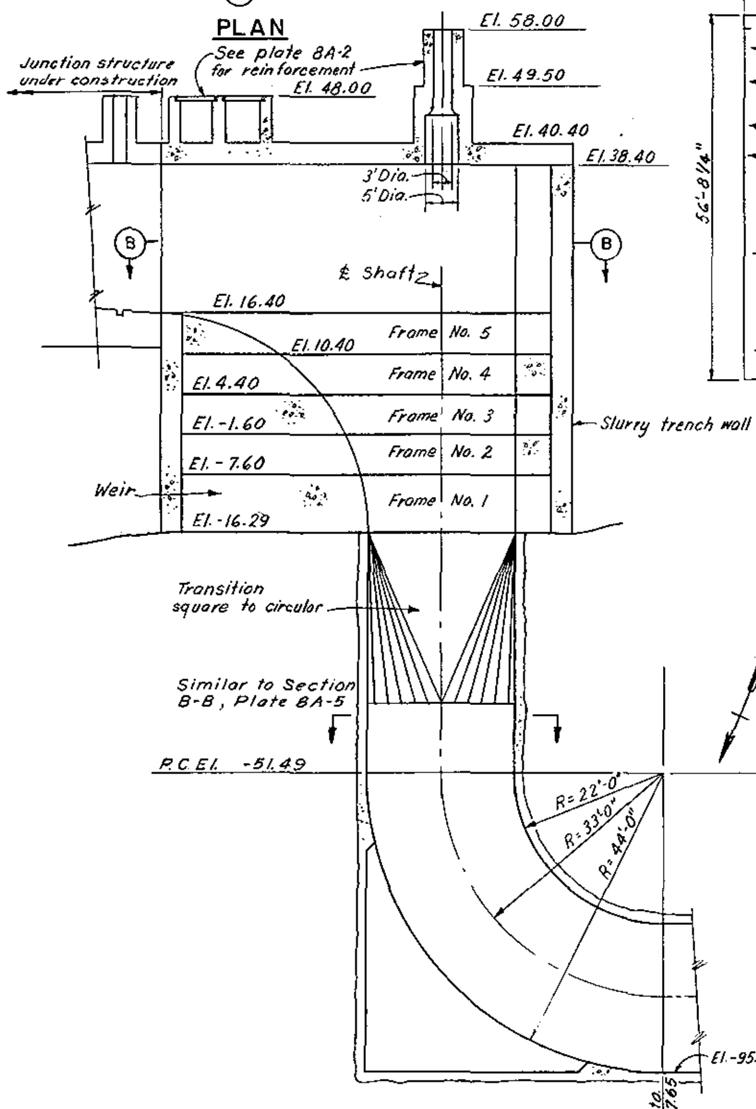
ROOF REINFORCEMENT PLAN
SCALE 3/32" = 1'-0"



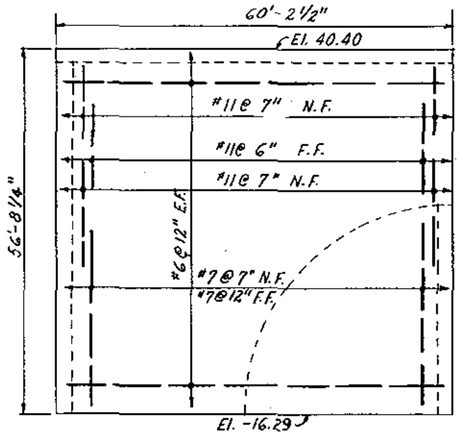
SECTION B-B
SCALE 3/32" = 1'-0"



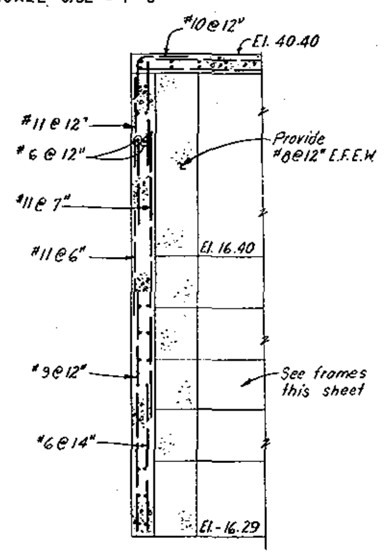
SECTION D-D
SCALE 3/32" = 1'-0"



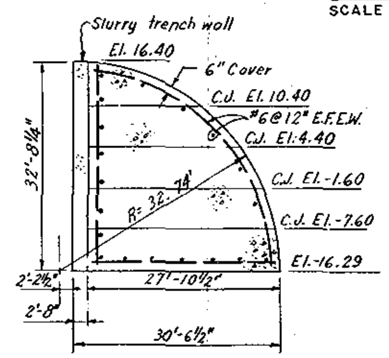
SECTION A-A
SCALE 1" = 10'-0"



SECTION C-C
SCALE 3/32" = 1'-0"

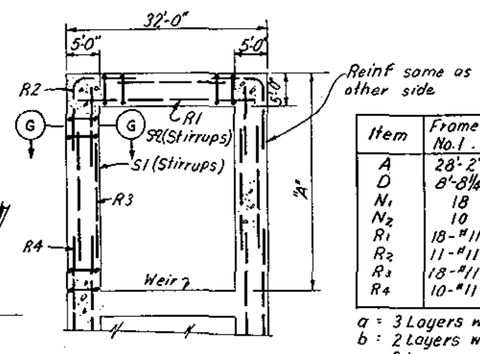


SECTION E-E
SCALE 3/32" = 1'-0"



WEIR REINFORCEMENT
SCALE: 1" = 10'

NOTES:
1. Construction procedure
   a. Install slurry trench wall @ conduit opening.
   b. Excavate and install internal bracing.
   c. Construct frames & weir, removing bracing as necessary.
   d. Construct remainder of intake structure.
2. Loading diagrams similar to those shown on Plate 8A-2.



PLAN OF FRAME
SCALE 3/32" = 1'-0"

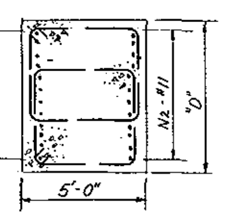
Item	Frame No. 1	Frame No. 2	Frame No. 3	Frame No. 4	Frame No. 5
A	28'-2"	30'-5"	34'-4"	40'-9"	57'-7"
D	8'-8¼"	6'-0"	6'-0"	6'-0"	6'-0"
N_1	18	12	12	12	12
N_2	10	13	13	36	26
R_1	18-#11	12-#11	12-#11	12-#11	12-#11
R_2	11-#11	6-#11	6-#11	12-#11	24-#11b
R_3	18-#11	12-#11	12-#11	12-#11	12-#11
R_4	10-#11	13-#11	13-#11	36-#11a	26-#11c

a = 3 Layers with 12 bars per layer
b : 2 Layers with 12 bars per layer
c = 2 Layers with 13 bars per layer

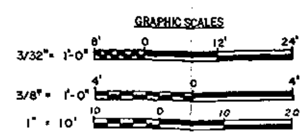
FRAME STIRRUPS

Frame No.	S1	S2
1	11@ 24"	12@ 24"
2	13@ 24"	12@ 24"
3	14@ 24"	12@ 24"
4	38@ 3" North 14@ 6" 7@ 24" 1@ 21" South	12@ 24"
5	12@5½" North 4@ 8" 3@12" 10@ 24" 2@12" 3@ 8" 26@ 4" South	9@4" East 4@8" 5@ 20" 4@ 8" 9@4 west

NOTE:
All stirrups to be #6 w/4 legs



SECTION G-G
SCALE 3/8" = 1'-0"



REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

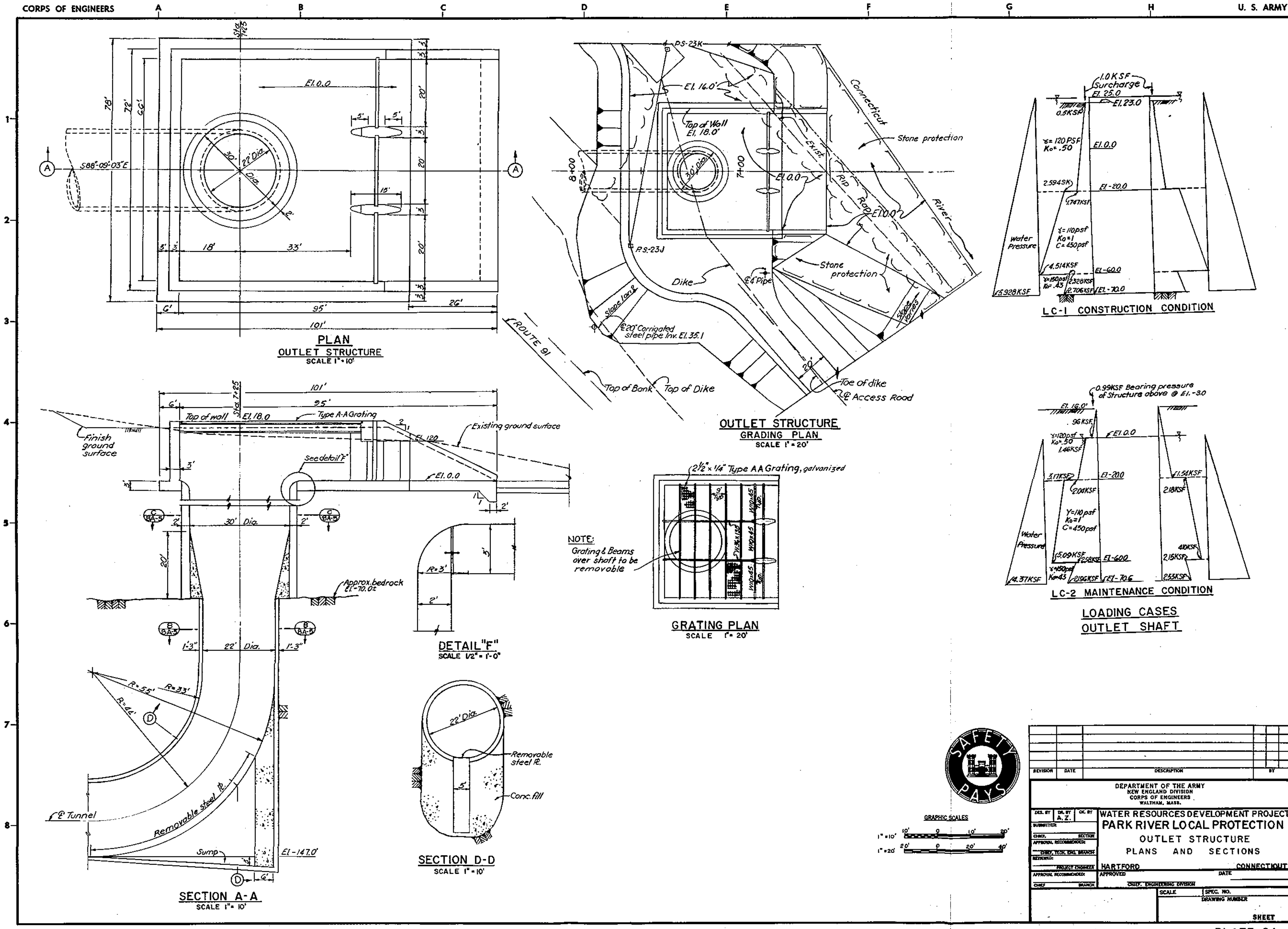
DES. BY | DR. BY | CK. BY
SUBMITTED: CHIEF, SECTION
APPROVAL RECOMMENDED: CHIEF, TECH. ENG. BRANCH
REVIEWED: PROJECT ENGINEER
APPROVAL RECOMMENDED: CHIEF BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
SLURRY WALL COFFERDAM SCHEME
INTAKE STRUCTURE
PLANS AND STRUCTURAL DETAILS
HARTFORD CONNECTICUT

APPROVED DATE
CHIEF, ENGINEERING DIVISION

SCALE AS SHOWN | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 8A-3



CORPS OF ENGINEERS
U. S. ARMY
PLAN
OUTLET STRUCTURE
SCALE 1" = 10'
S88°-09'-03"E
El. 0.0
22' Dia.
30' Dia.
101'
95'
OUTLET STRUCTURE
GRADING PLAN
SCALE 1" = 20'
PS-23K
PS-23J
El. 16.0'
Top of Wall
El. 18.0'
Connecticut
River
Stone protection
Exist. Rip Rap
Dike
Top of Bank
Top of Dike
Toe of dike
℄ Access Road
ROUTE 91
℄ 20" Corrugated steel pipe Inv. El. 35.1
LC-1 CONSTRUCTION CONDITION
1.0 KSF Surcharge
Water Pressure
LC-2 MAINTENANCE CONDITION
0.99KSF Bearing pressure of Structure above @ El. -3.0
LOADING CASES
OUTLET SHAFT
SECTION A-A
SCALE 1" = 10'
Top of wall El. 18.0
Type A-A Grating
Existing ground surface
Finish ground surface
See detail "F"
Approx. bedrock El. -70.0±
R = 55'
R = 33'
R = 44'
℄ Tunnel
Removable steel Pl.
Sump
El. -147.0'
DETAIL "F"
SCALE 1/2" = 1'-0"
SECTION D-D
SCALE 1" = 10'
Removable steel Pl.
Conc. fill
2 1/2" x 1/4" Type AA Grating, galvanized
NOTE:
Grating & Beams over shaft to be removable
GRATING PLAN
SCALE 1" = 20'
SAFETY PAYS
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
OUTLET STRUCTURE
PLANS AND SECTIONS
HARTFORD
CONNECTICUT

PLATE 8A-4

NEDED-E (30 Sep 76) 2d Ind
SUBJECT: Park River Local Protection, Connecticut River Basin, Hartford, Connecticut, DM No. 8, Auxiliary Conduit Shafts, Site Geology, Foundations and Materials and Detailed Design-Structures

DA, NED, CE, Waltham, Mass. 02154 14 January 1977

TO: DA, Office of the Chief of Engineers, Washington, D.C. 20314
ATTN: DAEN-CWE-B

The following actions and clarifications are presented in response to referenced comments:

a. Paragraph 2. Paragraph 5 of the Design Memorandum has been expanded to include an assessment of the effects of local drainage inflow on the flood control capacity of the auxiliary conduit. Pages 5 and 6 of the memorandum are deleted and copies of revised pages 5 and 6 are inclosed.

b. Paragraph 3. We concur - the contract specifications will require the necessary restraints and monitoring during construction.

c. Paragraph 4. The question of constructability is understood to pertain mainly to construction of the portion of the outlet shaft in soil. Previous discussion of alternate methods of constructing this shaft is contained in Appendix C of Design Memorandum No. 2, Phase II, Part II. In response we have revised and amended Paragraph 18. b. (1) to describe an alternate method of constructing the shaft. Both of the methods would be allowed by the contract plans and specifications. The shafts will be bid lump sum. A sketch showing the slurry wall method together with typical design computations are added to Appendix "C" (sheets DM8-78 to DM8-85) copies of which are inclosed. Pages 21 and 22 of the memorandum are deleted and are replaced by pages 21, 21a and 22, copies of which are inclosed.

d. In addition to the actions cited above, the following deletions and additions to data presented in Design Memorandum No. 8 are noted:

(1) Paragraph 17. c(2) has been revised to add rock loading to loading Case 5 for design of shaft linings in rock. In addition, the

rock modulus, 1,000,000 pounds per cubic inch, given in paragraph 17.d(2) is changed to read 1,000,000 pounds per cubic foot. Pages DM8-72 thru DM8-77 of Appendix C have been revised accordingly, copies of which are inclosed.

(2) Minimum cover (4 inches) for "all other surfaces" in paragraph 14.c.(4) is changed to 3 inches.

FOR THE DIVISION ENGINEER:

4 Incl (10 cys)
as

GEORGE T. SARANDIS
Acting Chief, Engineering Division

b. Outlet Structure. The configuration and function of the outlet structure has changed from those set forth in Design Memorandum No. 2. The outlet shaft in addition to being a working shaft for tunnel construction will now also serve as a clean out and inspection shaft for the completed Auxiliary Conduit. The radius of curvature at the intersection between the shaft and tunnel has been reduced and a sump area provided in the tunnel at the bottom of shaft. The sump will provide a flat invert at the bottom of the shaft for landing equipment used in clean out operations. Steel plates curved to the intersection radius will cover the sump when the conduit is in use. The portion of the shaft in overburden has been increased in diameter to 30 feet to compensate hydraulically for the reduced radius at its intersection with the tunnel. It also provides a seat on competent rock outside the area to be excavated for the shaft in rock. The outlet shaft has been separated from the outlet box, located at the top of the shaft, by a waterstopped joint to allow for differential movement and to prevent longitudinal bending that may be induced in the shaft by varying loading conditions at the outlet box. Similar to the intake shaft the outlet shaft will be excavated to the tunnel invert and the intersection radii formed with fill concrete. Additional features in the outlet structure include a removable open grating platform on top of the outlet box for safety of the public.

c. Clean Out and Inspection Shaft. The clean out and inspection shaft proposed at station 37+00 of the Auxiliary Conduit in the project design memorandum has been deleted from the project. Studies indicate that the shaft is not required for economical tunnel construction and its inclusion would increase the cost of the project by approximately two million dollars. The primary function of the deleted shaft has been incorporated into the outlet structure as described in the preceeding paragraph. It is concluded that the advantages of having an additional access shaft in the completed tunnel do not warrant the expenditures required.

d. Local Drainage Shafts. The Metropolitan District of Hartford has plans to separate the storm drains and sanitary sewers, which are presently combined, in the area South of Park Street. It would be more economical for them to discharge the storm water runoff into the auxiliary conduit via shafts than to conduct it to pumping stations at the Park River conduit for discharge. The District has requested that shafts be provided at the intersection of Park and Broad Streets and at the location where the auxiliary conduit crosses Main Street. The two 4-foot diameter shafts will be constructed as part of this project but will be non-Federal costs. The locations of the proposed shafts are shown on Plate 8A-1.

C. HYDROLOGY AND HYDRAULICS

5. GENERAL. The hydrology for the project was presented in Design Memorandum No. 1. The hydraulic analysis and results of model studies made were presented in Design Memorandum No. 3. A determination has been made that the changes in configuration of the inlet and outlet shafts will not significantly alter flow characteristics in the Auxiliary Conduit.

An assessment of local drainage inflows on the flood control capacity of the auxiliary conduit was made using the following two hydrologic conditions for analysis:

Case I — A 1-inch per hour rainfall applied to drainage areas served by the drop inlet shafts coincident with the project design flood flow rate in the auxiliary conduit and the 100-year frequency elevation in the Connecticut River.

Case II — A 25-year frequency rainfall was applied to the local drainage areas coincident with the 100-year frequency discharge in the auxiliary conduit and the 100-year elevation in the Connecticut River.

Under Case I conditions, the Connecticut River elevation is 30 feet msl and the design project flood flow rate of 5,400 cfs is augmented by discharges of 340 cfs and 210 cfs from the Broad Street and Main Street drop inlet shafts, respectively. Hydraulic analysis with the total Q = 5,950 cfs, results in the determination of an energy grade line (EGL) elevation at the Park River Junction Structure equal to 50.4 feet msl. This compares favorably with the value of 50.2 reported in D.M. No. 3, Hydraulic Analysis. The reason that the additional 550 cfs of local runoff only raises the EGL by 0.2 feet is that changes made in the vertical and horizontal alignments (deepening and straightening) of the auxiliary conduit and in the diameter (enlargement) of the outlet shaft since D.M. No. 3 was prepared have resulted in a reduction in total hydraulic losses from the formerly reported value of 20.2 feet to the present value of 18.4 feet.

Under Case II conditions, the 100-year peak flow rate in the auxiliary conduit is approximately 4,000 cfs and is augmented by a total local inflow of 875 cfs. The total peak Q = 4,875 cfs obviously results in a lower EGL at the Park River Junction Structure than that produced by the project design flow rate of 5,400 cfs.

The condition of project design storm rainfall producing peak rates of runoff from the local drainage areas served by the shafts coincident with the project design flow rate and tailwater in the auxiliary conduit was not analyzed since the local areas will have peaked several hours earlier than the time of the peak Park River flow into the auxiliary conduit.

D. CONCRETE

6. GENERAL. Concrete for the Auxiliary Conduit and shafts will be supplied from nearby commercial sources acceptable for use in civil works projects. Information on concrete was presented in Design Memorandum No. 4, Concrete Materials, Part I, Box Conduit. Any necessary updating or changes will be presented in Design Memorandum No. 9, Auxiliary Conduit Tunnel.

(2) Shaft Lining. The shaft lining was analyzed as a thin shell subjected to uniform internal and external pressures and lateral rock loads. Under loading cases 2 and 3 it was assumed that the rock interacts with the lining. A rock modulus of 1,000,000 pounds per cubic inch was assumed in the ring analysis when considering rock loading. An additional criterion was applied requiring a minimum ratio of area of transverse reinforcement to area of concrete of .0025. It was this criterion which determined the amount of transverse reinforcement specified for the circular shaft. To allow for required cover of reinforcement and proper placement of the concrete, a lining thickness of 15 inches was selected. The square and transition sections of the shaft will be more heavily reinforced.

18. OUTLET STRUCTURE.

a. General. The outlet structure comprises the outlet box concrete apron and a vertical circular shaft partly through earth and partly through rock. The outlet box will direct the discharge from the shaft to the river while preventing erosion of the surrounding soil. Stop logging features of the box will provide for dewatering of the shaft and tunnel during maintenance and inspection operations. The shaft will have an inside finished diameter of 30 feet from the invert of the outlet box to elevation of sound rock, a depth of approximately 70 feet. The shaft will transition from 30 feet to 22 feet at the top of the rock and continue through the rock with a finished inside diameter of 22 feet to the tunnel invert, a depth of approximately 77 feet. The outlet box will measure 66 feet by 95 feet in plan inside and will be founded on existing granular fill material some 15 feet below existing ground level. The remaining granular soil mantle overlying the clay will be 15 feet thick under the invert slab. To allow for differing foundation conditions between the shaft and outlet box, a waterstopped joint will be provided at their intersection. To provide protection for the shaft and tunnel against flooding during construction an earthen berm will be constructed to elevation 25.0 and the shaft constructed through it. The top of the shaft will be maintained a minimum of 2 feet above berm elevation until completion of the tunnel, thereby providing protection against the theoretical 40 year storm. The berm will be large enough to provide adequate work area for construction operations and those portions not required in final grading of the project will be removed after completion of the outlet structure.

b. Construction Procedure.

(1) Shaft in Soil. Alternate methods of constructing the portion of the outlet shaft in soil will be allowed by the contract specifications. Two of the possible methods of constructing the shaft are as follows:

(a) Reinforced Concrete Sunk Caisson. Under this alternate the shaft will be cast above the earth berm in lifts and sunk, open ended, to the rock surface below. The shaft will be sunk by excavating in the wet with jetting outside as necessary to reduce frictional resistance between the shaft and soil. The bottom of the shaft will be contoured to match the shape of the rock surface to minimize the problem of sealing the interface of the shaft and rock surface during dewatering. The total height of shaft to be constructed in this manner, including the temporary portion, is 95 feet. It is felt that pneumatic methods will not be required during construction of the shaft. Grouting of the rock in the area immediately outside the bottom of the shaft will aid in sealing the flow of water and can be accomplished from the surface.

(b) Concrete Shaft Cast-in-Place by the Slurry Trench Method. This method will consist of excavating a trench for the shaft in 7 panels along its circumference and placing shaft concrete by tremie methods through a bentonite slurry which will maintain the sides of the trench. A minimum head of slurry of 6 feet above flood level will be required to insure against collapse of the excavated trenches prior to placement of concrete. Each panel will be formed by three excavated chords, the ends of which lie on the centerline circumference, approximately 5 feet in length. The ends of each concrete panel will be slip-formed by means of steel pipes to provide the keying mechanism with adjacent panels. The number and lengths of panels and finished diameter can be varied to suit contractors excavating equipment. Minimum shaft thickness, diameter and required surface finish will be specified. A sketch of one possible configuration of shaft placed by slurry method with representative design computations have been included in Appendix C.

(2) Shaft in Rock. Once the shaft in earth is constructed, properly seated on rock and the flow of water controlled, excavation for the shaft in rock can begin. The top 20 feet of the shaft in rock will require light excavating methods so that the shaft in soil is not disturbed. Bolting and shotcreting will closely follow the excavation to minimize water control and dangers of falling rock.

(3) Outlet Box. Upon completion of the shaft and tunnel, a braced sheet pile cofferdam will be constructed within the earthen berm for construction of the outlet box in the dry. The cofferdam sheeting will be driven deep enough to provide sufficient cutoff so that normal pumping will keep the water below the bottom of excavation during construction. After completion of excavation and dewatering operations the temporary shaft above elevation -3.0 will be removed and the outlet box constructed. After completion of the outlet box the earthen berm will be removed to the level of final grading, the cofferdam removed and final riprap protection placed.

c. Loading Conditions.

(1) Outlet Box. The three conditions of loading investigaed in the design of the outlet box are as follows:

Case 1. Construction Condition - Dead load of the concrete structure only in the dewatered cofferdam.

Case 2. Operating Condition - Full dead load, at rest lateral earth pressure and river level at the bottom of the invert slab.

Case 3. Maintenance Condition - Full dead load, at rest lateral earth pressure, no water inside and water outside the box to the top of the stop logs at elevation 12.0.

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

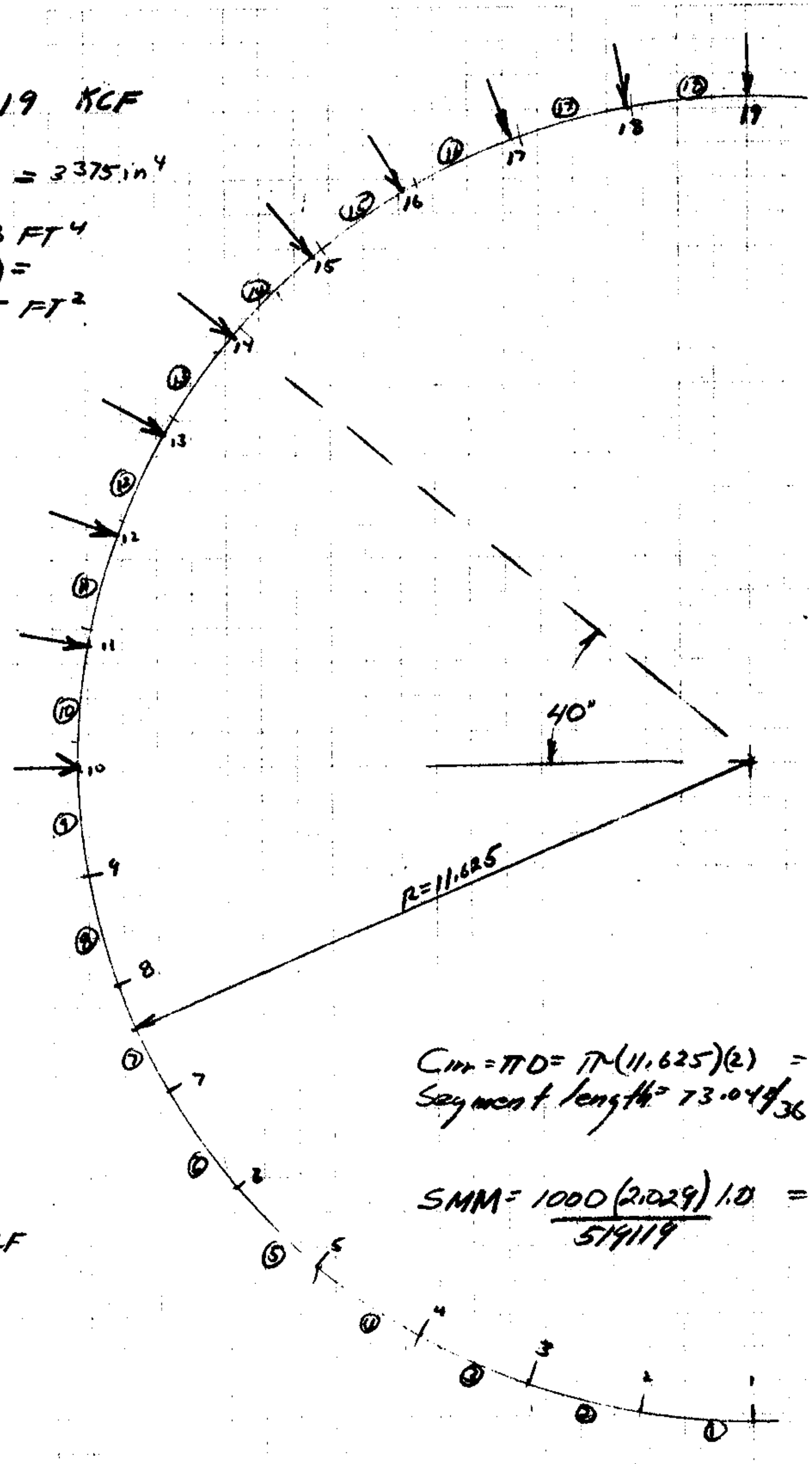
DM 8-72
(Revised 12 Jan 77)
PAGE

SUBJECT Park River - Auxiliary Conduit Shaft in Rock
COMPUTATION Computer Model
COMPUTED BY TKH CHECKED BY RLH DATE Dec 76

$E_c = 519119$ KCF

$I_{15} = \frac{15^3}{12} = 3375 \text{ in}^4$
$= .163 \text{ FT}^4$

$A_{15} = 15(12) =$
$= 1.25 \text{ FT}^2$



$C_{irr} = \pi D = \pi(11.625)(2) = 73.042'$
Segment length $= 73.042/36 = 2.029$

$SMM = \frac{1000(2.029)1.0}{519119} = .00390 \text{ FT}^2$
$\underline{\underline{.00195 \text{ FT}^2}}$

R= 1000 KCF

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

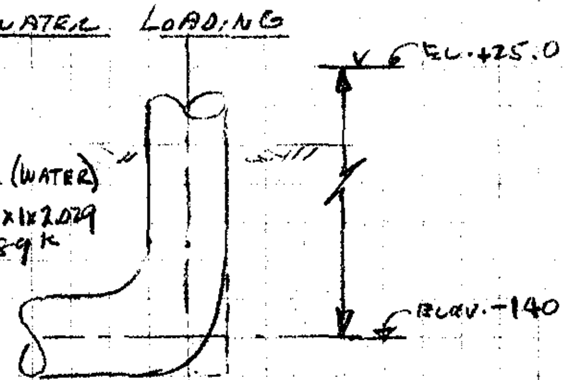
PAGE DM8-73 (Revised 12 Jan 77)

SUBJECT Park River – Auxiliary Conduit Shaft in Rock
COMPUTATION Loading Outlet Shaft in Rock
COMPUTED BY TKH CHECKED BY RLK DATE JAN 77

Rock Loads $0.68 T/FT^2 = 1.36\ K/FT^2$

Node	Load Y KIPS
10	.060
11	.478
12	.938
13	1.378
14	1.736
15	2.110
16	2.386
17	2.590
18	2.714
19	1.378

WATER LOADING



NODE LD. (WATER)
$165 \times .0624 \times 1 \times 2.029$
$= 20.89^K$

NODE	LC-2 Rock Load Only X	LC-2 Rock Load Only Y	External Radial	Water Construction Condition X	Water Construction Condition Y	LC-5 External + Rock X	LC-5 External + Rock Y
1	0	0	10.45	0.0	10.47	0.0	10.47
2	0	0	20.89	3.63	20.57	3.63	20.57
3	0	0	↓	7.14	19.63	7.14	19.63
4	0	0		10.45	18.09	10.45	18.09
5	0	0		13.43	16.00	13.43	16.00
6	0	0		16.00	13.43	16.00	13.43
7	0	0		18.09	10.45	18.09	10.43
8	0	0		19.63	7.14	19.63	7.14
9	0	0		20.57	3.63	20.57	3.63
10	0	-.060		20.89	0.0	20.89	-.060
11	0	-.478		20.57	-3.63	20.57	-4.108
12	0	-.938		19.63	-7.14	19.63	-8.078
13	0	-1.378		18.09	-10.45	18.09	-11.828
14	0	-1.736		16.00	-13.43	16.00	-15.166
15	0	-2.110		13.43	-16.00	13.43	-18.110
16	0	-2.386		10.45	-18.09	10.45	-20.476
17	0	-2.590		7.14	-19.63	7.14	-22.220
18	0	-2.714	20.89	3.63	-20.57	3.63	-23.284
19	0	-1.378	10.45	0.0	-10.45	0.0	-11.828

LC-2 ROCK LOAD ONLY
OUTLET SHAFT .63T/ft² in ROCK
DM 8-74
(Rev 12 Jan77)

MEMBER FORCES (LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	.01495	1.12583	17.24533
2	.03287	1.19345	17.24211
3	.05343	1.30175	17.23461
4	.03010	1.45403	17.22234
5	.05453	1.57453	17.21114
6	-.03272	1.50323	17.20921
7	-.19223	1.11356	17.22902
8	-.48333	.13845	17.23310
9	-.37038	-1.52650	17.40543
10	-1.20776	-4.07443	17.52844
11	-1.34113	-5.79147	17.27343
12	-.96345	-8.74405	16.59512
13	.47155	-7.73357	15.44051
14	1.90977	-3.91839	13.39324
15	2.55909	1.25546	12.14563
16	2.47615	6.23479	10.50515
17	1.76019	9.85030	9.24705
18	.53241	11.13254	3.56510

BLOCKING POINT THRUSTS

JOINT	THRUST
1	1.49126
2	2.98189
3	2.98277
4	2.99057
5	3.01719
6	3.07744
7	3.17445
8	3.29727
9	3.40325
10	3.33553
11	3.16029
12	2.57448
13	1.37559
14	1.11523
15	1.51600
16	2.06616
17	2.43402
18	2.57276
19	1.37300

ND JOINT REACTIONS

OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-17.13225	-1.09657	.00000
LAST	-8.47751	11.13254	1.37300

RMD014 W INS EOF
WALKBACK SEQUENCE

1?	.00045	-.00071	-.01344	.01273
?	.00035	-.00039	-.01454	.01443
1?	0.	0.	-.01503	.01503

LC-5 CONSTRUCTION Condition
Rock LOAD + External Water 0.63T/FT²
OUTLET SHAFT in ROCK
DM8-75
(Rev 12 JAN 77)

MEMBER FORCES LOCAL COORDINATES)

MEMBER	SHEAR	MOMENT	THRUST
1	.03535	2.45520	135.45531
2	.05797	2.57363	135.44517
3	.09913	2.77454	135.43122
4	.22071	3.22170	135.40100
5	.07319	3.37001	135.37775
6	-.23730	2.88923	135.38971
7	-.59582	1.68212	135.46504
8	-1.31435	-.98170	135.63234
9	-2.08205	-5.20150	135.94935
10	-2.56507	-10.40024	135.29650
11	-2.19019	-14.83743	135.23523
12	.15411	-14.52508	135.52919
13	2.19322	-10.08151	134.12293
14	3.34023	-3.31232	132.30814
15	3.55519	4.09324	130.33339
16	3.35291	10.90858	128.52117
17	2.30707	15.58255	127.13593
18	.79257	17.18899	125.39624

BLOCKING POINT THRUSTS

JOINT	THRUST
1	11.79449
2	23.53995
3	23.55500
4	23.59404
5	23.67295
6	23.83432
7	24.07408
8	24.33177
9	24.33595
10	24.25700
11	23.33092
12	21.38807
13	21.53058
14	22.00558
15	22.50535
16	22.95756
17	23.32221
18	23.56060
19	11.82300

ND JOINT REACTIONS

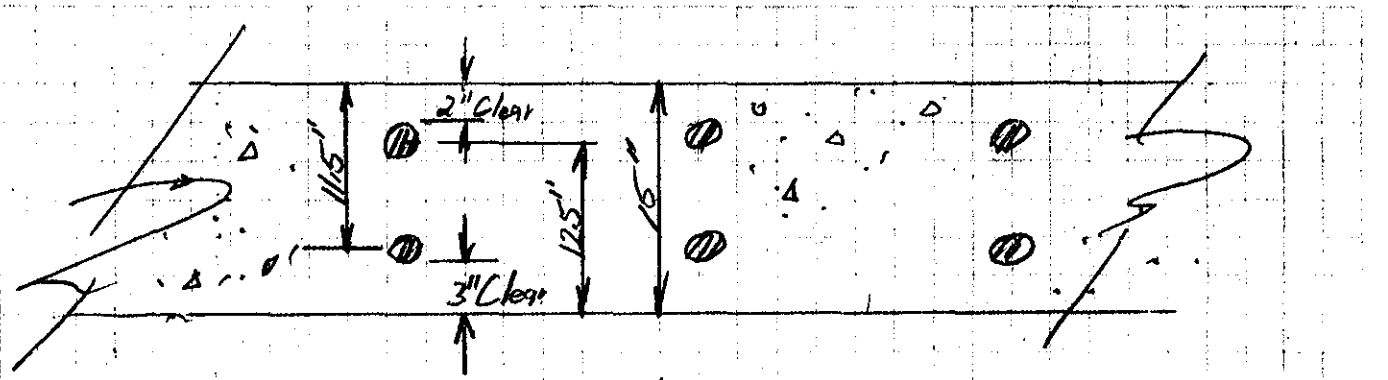
OINT	X-FORCE	MOMENT	Y-FORCE
FIRST	-134.94584	-2.38455	-10.47000
LAST	-125.84309	17.18899	11.82799

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

DM8-76
(Revised 12 Jan 77)
PAGE

SUBJECT: Park River – Auxiliary Conduit Shaft
COMPUTATION: Cast in Place 15" Liner Outlet Shaft in Rock
COMPUTED BY: JKH CHECKED BY: RLH DATE: Jan 77



Positive Moments node 19			Negative Moments node 3		
$d = 11.5''$ $d'' = 4''$ $d' = 2.5$	M	N	$d = 12.5''$ $d'' = 5''$ $d' = 3.5''$	M	N
LC-2	$11.13^{'K}$	8.56^{K}	LC-2	$-8.74^{'K}$	16.59^{K}
LC-5	$17.19^{'K}$	126.310^{K}	LC-5	$-14.84^{'K}$	136.24^{K}

LC-2 node 19 $M = 11.13^{'K}$ $N = 8.56^{'K}$ (critical Loading)

Avg Compression

$$f_{c\,Avg} = \frac{8.56(1000)}{15(12)} = 47.56 \text{ psi}$$

Max & Min Compression

$S = \frac{15^3}{12} \cdot \frac{2}{15} = 450$

$$f_c = \frac{8.56(1000)}{15(12)} \pm \frac{11.13(12000)}{450}$$

$= 47.56 \pm 296.80$
$= 344.36$ psi Compression
$\;\;\;249.24$ psi Tension

$e = 12M/N = 12(11.13)/8.56$
$= 15.60$

$e_0 = 15.60 - 15/2 = 7.50$ Try #6 @ 12 e.f. $A_s = A_s' = .44$

$Kd^3 + 22.50Kd^2 + 66.44Kd - 467.06 = 0$

Try $Kd = 4$ (222.70)
Try $Kd = 3$ (−38.24)   Try 3.15 (−3.26)
Try 3.20 (8.72)   Say $Kd = 3.16$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

DM8-77
(Revised 12Jan77)

SUBJECT Park River - Auxiliary Conduit Shaft
COMPUTATION Cast in Place 15" Liner Outlet Shaft in Rock
COMPUTED BY TAH CHECKED BY RLLL DATE Jan 77

$$f_c = \frac{8.5^6}{12\frac{(3.16)}{2} + \left(\frac{3.16-2.5}{3.16}\right)15(.44) - \left(\frac{11.5-3.16}{3.16}\right)8(.44)} = \underline{774.78 \text{ psi}}$$

$$f_s = \left(\frac{11.5-3.16}{3.16}\right)8(774.78) = \underline{16358.65 \text{ psi}}$$

$$f_s' = \frac{3.16-2.5}{3.16}(16)(774.78) = \underline{2581.14 \text{ psi}}$$

Use $\boxed{\#6@12 \text{ EF}}$

Check Avg Comp LC-5 $N = 136.24$

$$f_{c\,avg} = 136.24/15(12) = \underline{756.86 \text{ psi ok}}$$

Max & Min Compression LC-5 $M = 17.19$ $N = 126.21$

$$f_c = 126.21/15(12) \pm \frac{17.19(12)}{450}$$

$$= 1159.57 \text{ psi Compression}$$
$$\underline{242.77 \text{ psi Compression}} \quad \text{ok}$$

Check Principal Stress

LC-1 note 16 $V = 2.56$ $N = 12.15$

$d = 11.5$
$A = 11.5(12) = 138$

$$v_c = \frac{2.56(1000)}{138} = 18.55 \qquad f_c = \frac{12.15(1000)}{138} = 88.04$$

$$f_t = \frac{88.04}{2} \pm \sqrt{\left(\frac{88.04}{2}\right)^2 + 18.55^2} =$$

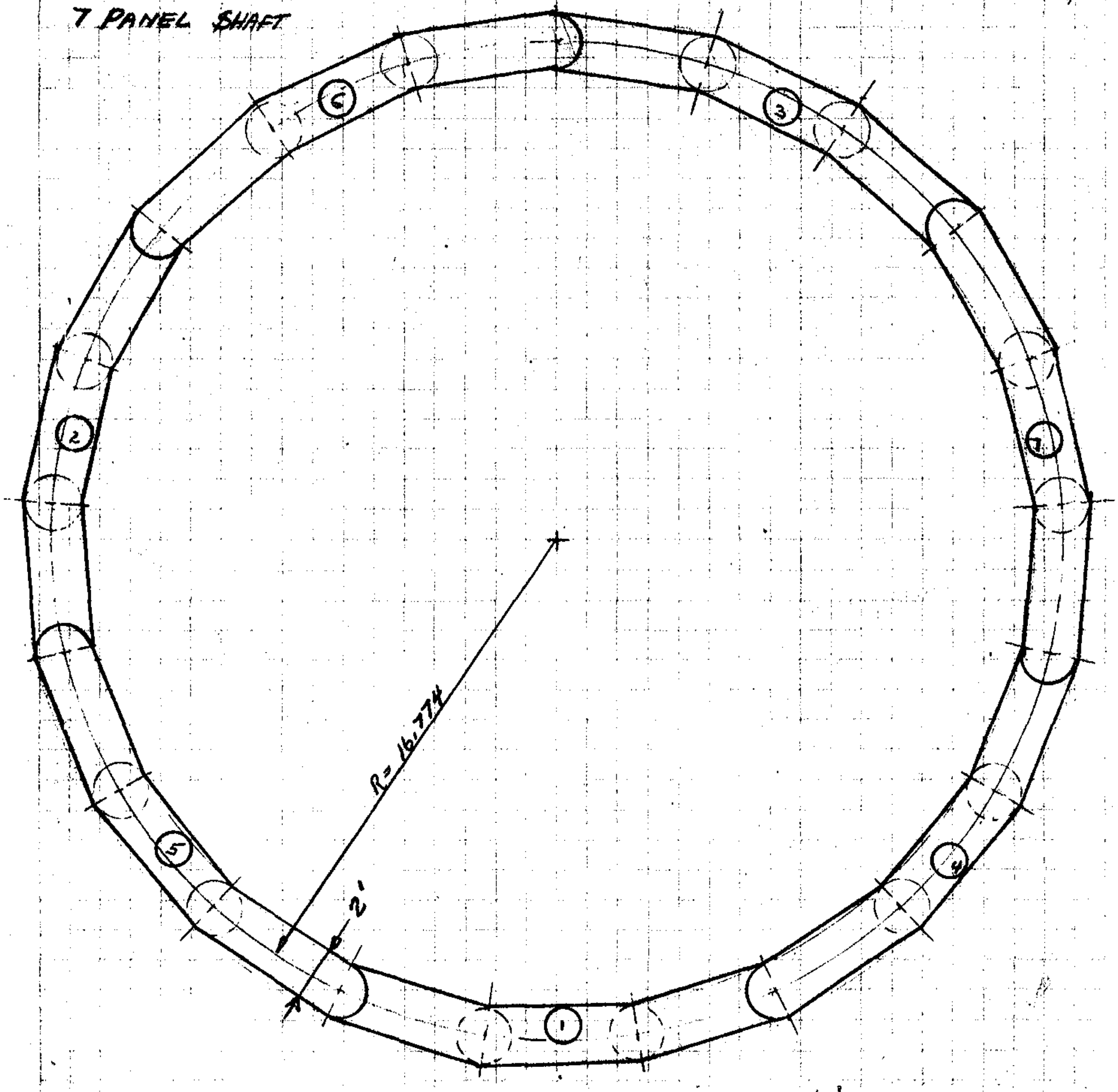
$$44.02 \pm 47.77 = 3.75 \text{ psi Tension}$$
$$91.79 \text{ psi Comp. ok}$$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Park River- Auxiliary Conduit Shaft in Soil
COMPUTATION Computer Model
COMPUTED BY TKH CHECKED BY RLH DATE Dec 76

3 SLOT PANELS
7 PANEL SHAFT



21 segments Cord length = 5'
$360/21 = 17.14°$

$r = 16.774'$

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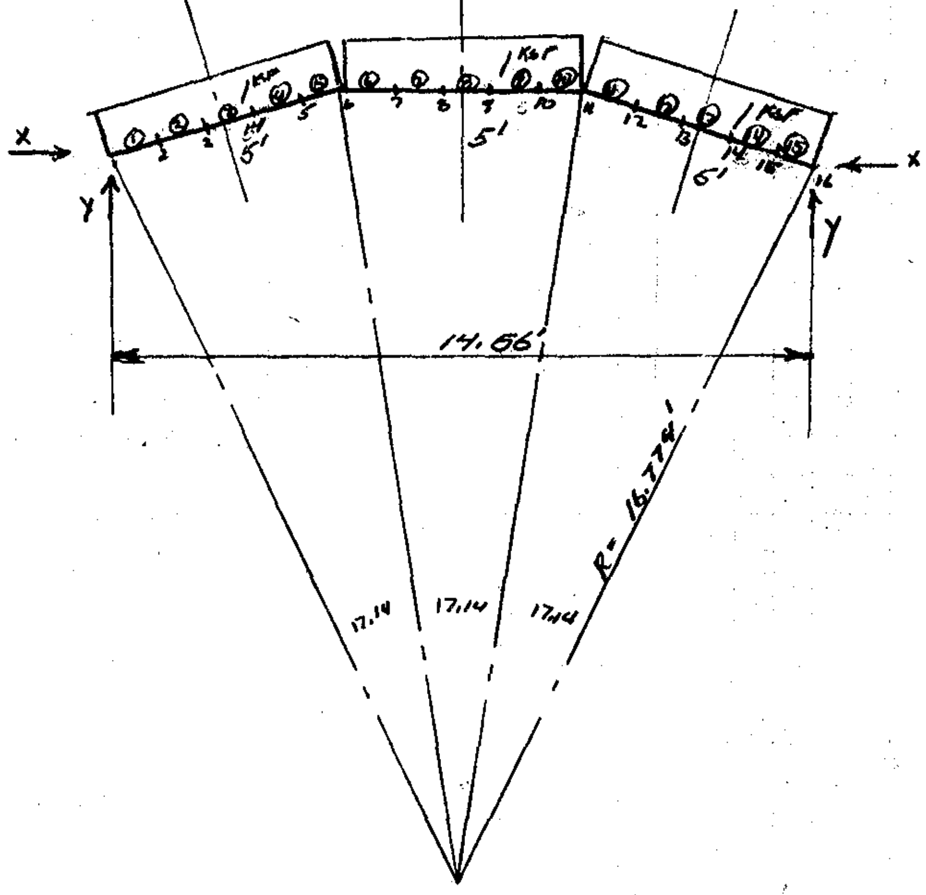
PAGE DM8-79

SUBJECT Park River - Auxiliary Conduit Shaft in Soil
COMPUTATION Three Segment Pannel Unit Load
COMPUTED BY TKH CHECKED BY RLH DATE Jan 77

Analysis - Based on unit load of 1.0 ksf



Summary of Results

	Node	Axial	Shear	Moment
MAX MOMENT	8, 9	13.60	.5	6.197
MAX SHEAR	1, 16	14.66	2.89	0
MAX AXIAL	1, 16	14.66	2.89	0

MAX MOMENT OCCURS AT MID SPAN OF ELEMENT 8

Axial	Shear	Moment
13.60	0	6.322

14	-.22262E-04	-.16723E-03	.74713E-04
15	-.12045E-04	-.36915E-04	.34155E-04
16	0.	0.	.37377E-04

DN8-80

Shaft in EARTH - Unit LOADING
3 SLOT Pannels

END ACTIONS WITH ELEMENT LOADS

ELEMENT	AXIAL I / AXIAL J	SHEAR I / SHEAR J	MOMENT I / MOMENT J
1	14.663	2.887	-.000
	-14.477	-1.988	2.438
2	14.477	1.988	-2.438
	-14.292	-1.089	3.977
3	14.292	1.089	-3.977
	-14.106	-.190	4.616
4	14.106	.190	-4.616
	-13.920	.709	4.356
5	13.920	-.709	-4.356
	-13.734	1.608	3.197
6	13.600	2.500	-3.197
	-13.600	-1.500	5.197
7	13.600	1.500	-5.197
	-13.600	-.500	6.197
8	13.600	.500	-6.197
	-13.600	.500	6.197
9	13.600	-.500	-6.197
	-13.600	1.500	5.197
10	13.600	-1.500	-5.197
	-13.600	2.500	3.197
11	13.734	1.608	-3.197
	-13.920	-.709	4.356
12	13.920	.709	-4.356
	-14.106	.190	4.616
13	14.106	-.190	-4.616
	-14.292	1.089	3.977
14	14.292	-1.089	-3.977
	-14.477	1.988	2.438
15	14.477	-1.988	-2.438
	-14.663	2.887	.000

1

** EFFRAM ** ANALYSIS OF PLANE FRAMES ON ELASTIC FOUNDATIONS

** DATE 01/06/77
** TIME 10:50:44
SECTION 2

** END ***

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

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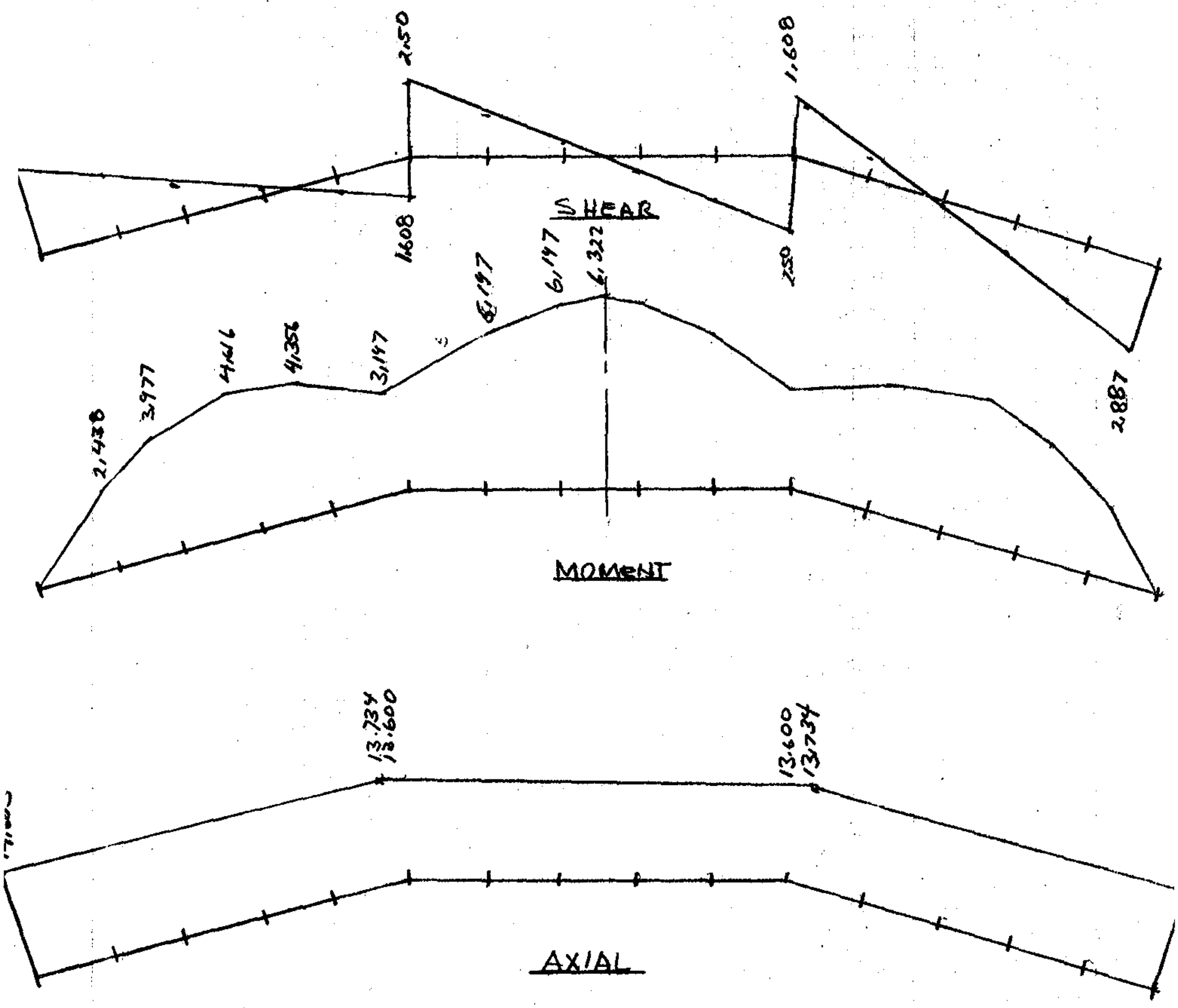
DM8-01
PAGE

SUBJECT Park River- Auxiliary Conduit Shaft in Soil

COMPUTATION Three Segment Pannel Unit Load

COMPUTED BY TKH CHECKED BY DATE Jan 77

FOR UNIT LOAD



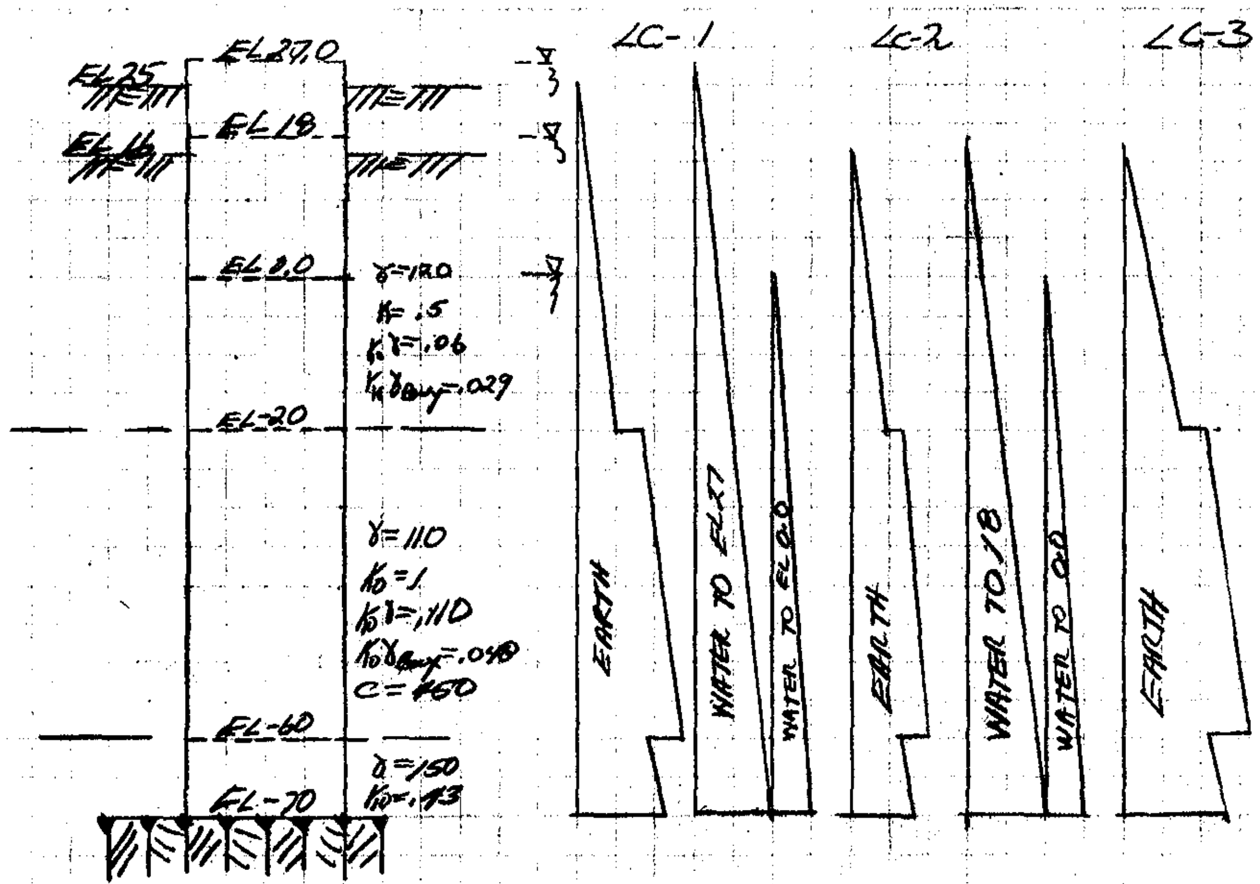
NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

SUBJECT Park-River – Auxiliary Conduit Shaft in Soil

COMPUTATION Outlet Shaft Loading Conditions

COMPUTED BY TKH CHECKED BY RLH DATE JAN 77



LC-1 Construction Condition Net External Water
LC-2 Maintenance Conditions Net External Water
LC-3 Operating Condition Balanced Water

ELEV	LC-1 Pressure KSF		LC-2 Pressure KSF		LC-3 Pressure
	Water to EL27	Water to EL 0.0	Water to EL18	Water to EL 0.0	No Water
27	0	0			
25	.12	0			
25	.12	0			
18			0	0	0
16			1.12	0	0
16			1.12	0	0
0	2.41	1.60	1.58	.96	.96
-20	4.24	3.33	3.41	2.79	1.54
-20	4.69	3.78	3.86	3.24	1.99
-35	6.35	6.43	5.52	4.89	2.71
-60	9.11	8.19	8.28	7.65	3.91
-60	8.66	7.74	7.83	7.20	3.46
-70	9.66	8.75	8.83	8.21	3.84

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

DMB-85
PAGE ____

SUBJECT Park River – Auxiliary Conduit Shaft in Soil
COMPUTATION Three Segment Pannel Design
COMPUTED BY TKH CHECKED BY RLH DATE JAN 77

1/3 Increase

Assume $f'_c = 3000$ psi
$f_c = 1350$ psi — 1800 psi
f_c(tension) = 88 psi — 117.33 psi

MAX DEPTH WHERE NO STEEL IS REQUIRED

TRY −35.00′ LC-7 P = 6.35

$M = 6.35(6.332) = 40.21$ 'K
$N = 6.35(13.60) = 86.36$ 'K

Avg Compression

$f_{c\,avg} = 86.36 / 12(24) \times 1000 = \underline{299.86 \text{ psi}}$

$S = 24^3 \frac{2}{24} = 1152$

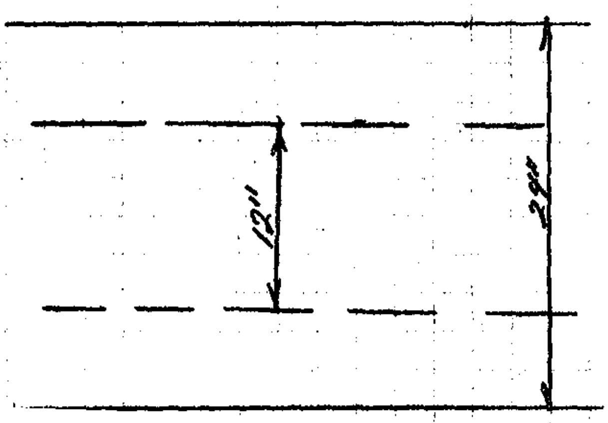
Max & Min Compression

$$f_c = \frac{86.36(1000)}{12(24)} \pm \frac{40.21(12000)}{1152}$$

$= 299.86 \pm 418.85$
$= 718.71$ psi Compression
118.99 psi Tension
≈ 117.33 psi allowable

All pannels below elevation −35′ will require some min steel reinforcement.

For Ease of Construction a 12″ steel cage is assumed this gives a max design depth of 12+6 = 18″



Critical load in Clay at −60 FT P = 9.11 KSF LC-1

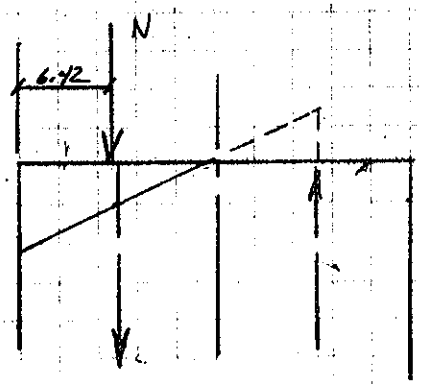
$M = 9.11(6.322) = 57.593$
$N = 13.6(9.11) = 123.896$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River - Auxiliary Conduit Shaft in Soil
COMPUTATION Three Segment Pannel Design
COMPUTED BY TKH CHECKED BY RLH DATE JAN 77

$d = 18 \quad d' = 6.5$



$e = \frac{12M}{N} = \frac{12(57.593)}{123.896} = 5.58$

$e_0 = 5.58 - 24/2 = -6.42$

$\Sigma M_N = 0$

$\frac{1}{2}(12)\,Kd\,(Kd/3 - e_0) + \left(\frac{Kd - d'}{Kd}\right)(2n-1)\,A_s'(d' - e_0) - \left(\frac{d - Kd}{Kd}\right) n\,A_s(d - e_0) = 0$

Try #6 @ 12 ef (.44)

$\frac{1}{2}\,12\,Kd\,(Kd/3 - 6.42) + \left(\frac{Kd - 6.5}{Kd}\right) 15(.44)(6.5 - 6.42) - \left(\frac{18 - Kd}{Kd}\right) 8(.44)(18 - 6.42) = 0$

$2Kd^2 - 38.52Kd + \left(\frac{.53Kd - 3.43}{Kd}\right) - \left(\frac{733.71 - 40.76Kd}{Kd}\right)$

$2Kd^3 - 38.52Kd^2 + 41.29Kd - 737.14 = 0$

$Kd^3 - 19.26Kd^2 + 20.65Kd - 368.57 = 0$

Try $Kd = 12$ (-1166.21)
Try $Kd = 18$ (-405.11)
Try $Kd = 20$ (340.43)

Kd is greater than 18" therefour minimum steel is all that is required since N fall inside the Kern

$f_c = \left(\frac{N}{bD} + \frac{6M(12)}{bD^2}\right) = \frac{123.896}{12(24)} + \frac{6(57.593)(12)}{12(24)^2}$

= 1030.12 psi Compression OK

$\boxed{\text{Use \#6 @ 12'' ef.}}$

NED FORM 223
27 Sept 49

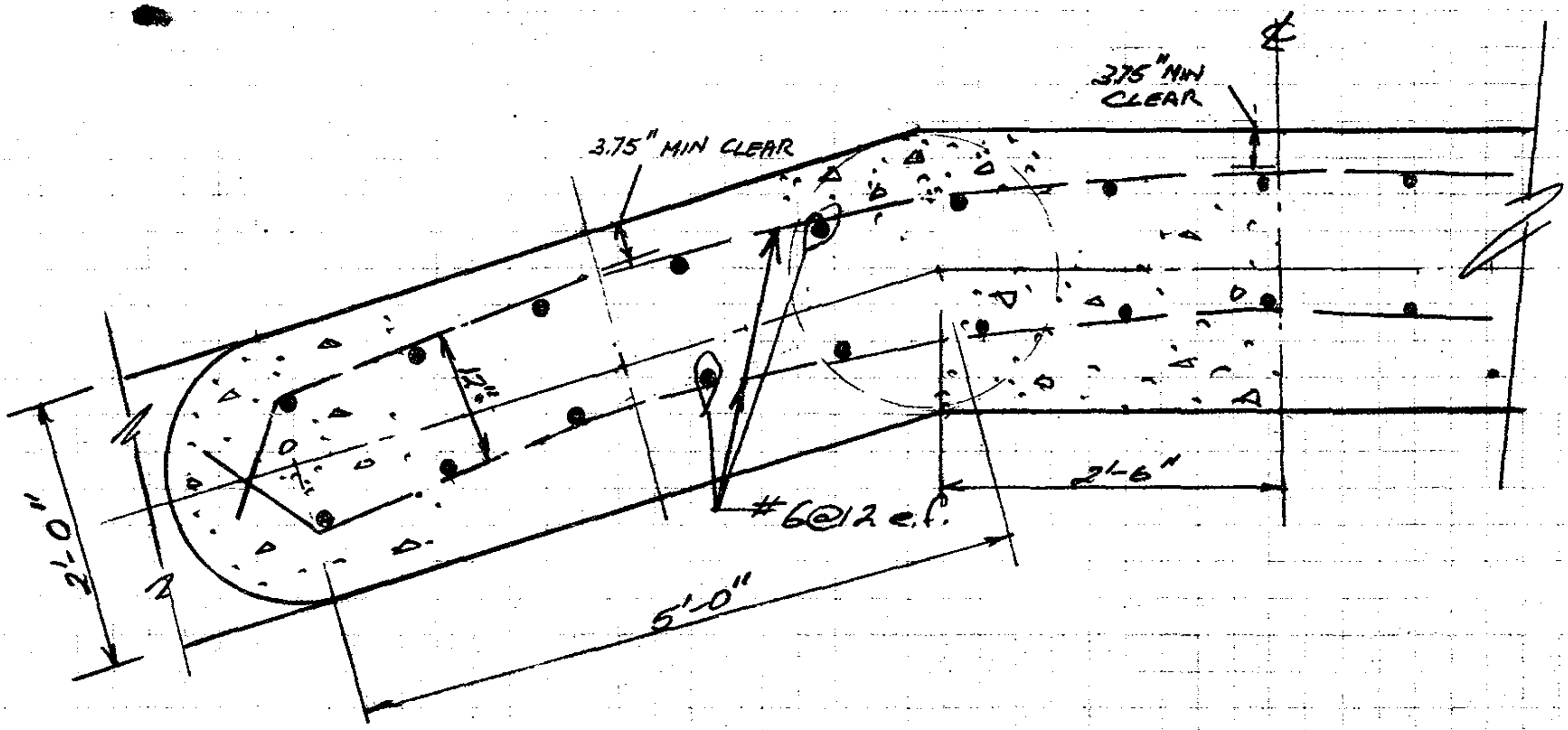
NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Park River – Auxiliary Conduit Shaft in Soil

COMPUTATION Three Segment Pannel Design

COMPUTED BY TMI CHECKED BY E.H.T DATE Jan 77

PAGE DMB-85



Typical Pannel Section

Scale 3/4" = 1'-0"

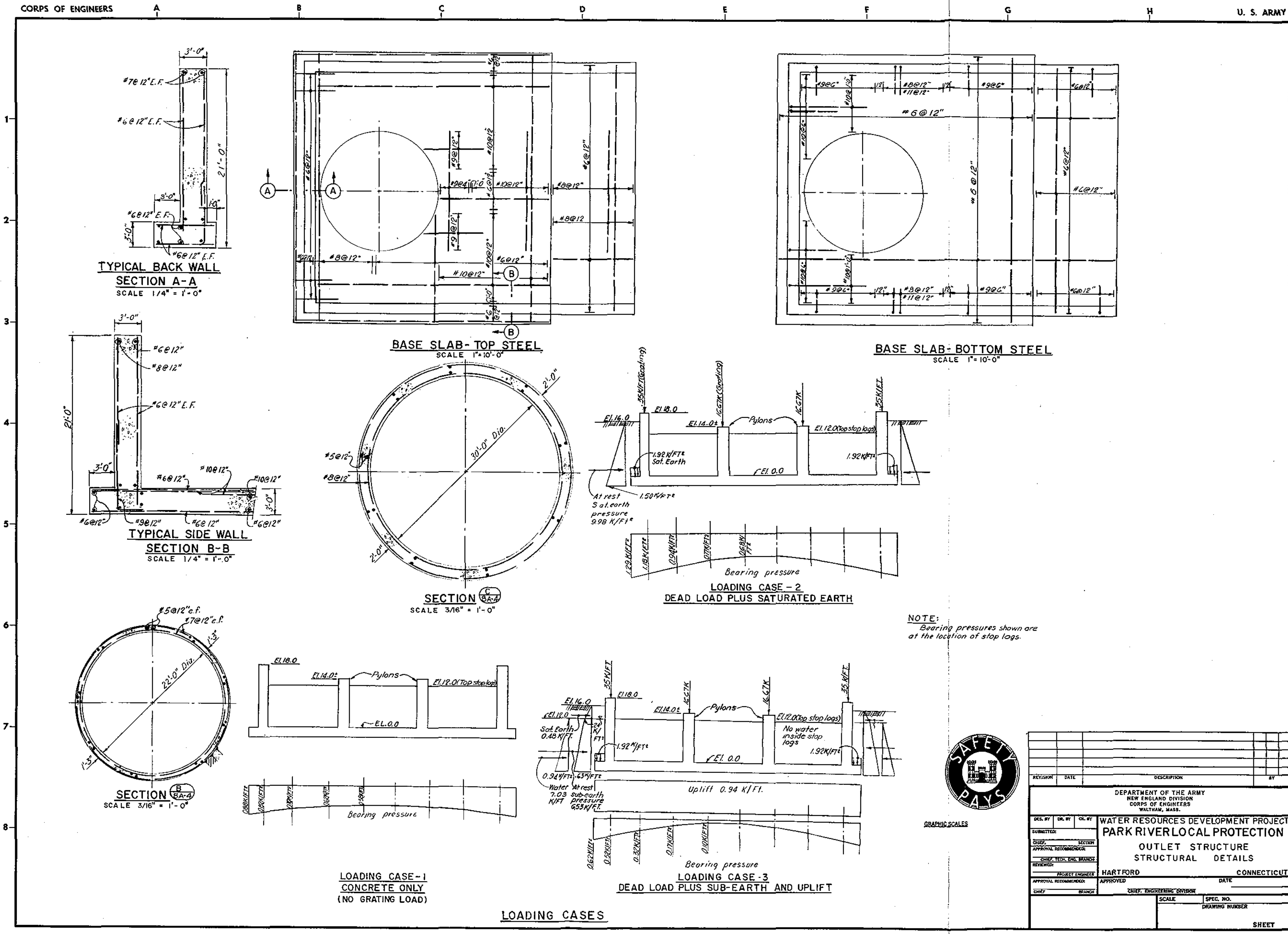
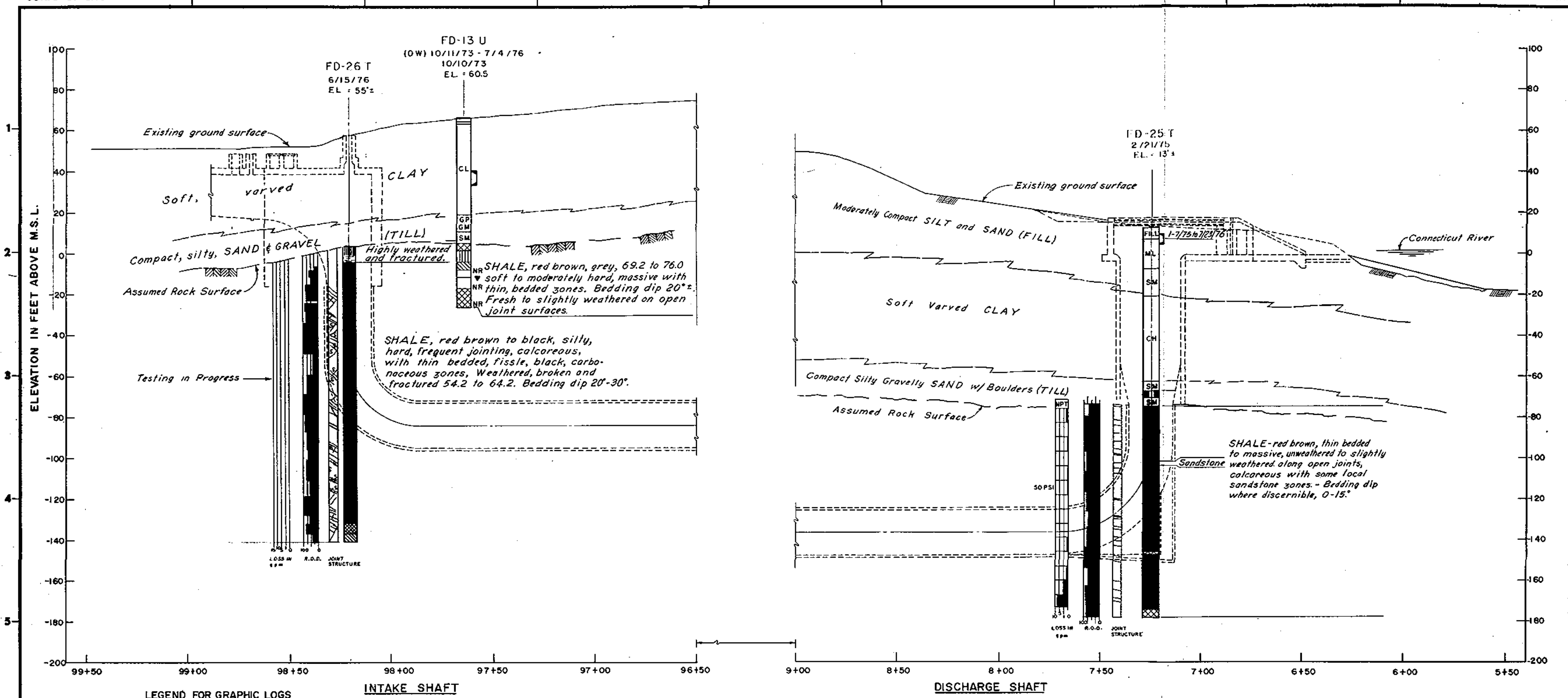

CORPS OF ENGINEERS
U. S. ARMY
TYPICAL BACK WALL
SECTION A-A
SCALE 1/4" = 1'-0"
BASE SLAB- TOP STEEL
SCALE 1" = 10'-0"
BASE SLAB- BOTTOM STEEL
SCALE 1" = 10'-0"
TYPICAL SIDE WALL
SECTION B-B
SCALE 1/4" = 1'-0"
30'-0" Dia.
SECTION C 8A-4
SCALE 3/16" = 1'-0"
Pylons
Bearing pressure
LOADING CASE - 2
DEAD LOAD PLUS SATURATED EARTH
NOTE:
Bearing pressures shown are at the location of stop logs.
22'-0" Dia.
SECTION B 8A-4
SCALE 3/16" = 1'-0"
Uplift 0.94 K/FT.
LOADING CASE - 1
CONCRETE ONLY
(NO GRATING LOAD)
LOADING CASE - 3
DEAD LOAD PLUS SUB-EARTH AND UPLIFT
LOADING CASES
SAFETY PAYS
GRAPHIC SCALES
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.
WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
OUTLET STRUCTURE
STRUCTURAL DETAILS
HARTFORD
CONNECTICUT

PLATE 8A-5

CORPS OF ENGINEERS | U. S. ARMY



LOG PROFILE

SCALE: 1" 20'

LEGEND FOR GRAPHIC LOGS

FD-610	TYPE AND NUMBER OF EXPLORATION.
(OW)1/7-15/54	OBSERVATION WELL INSTALLED AND PERIOD OF OBSERVATION.
N15,000 W9,876	LOCATION BY COORDINATE.
1/7/54	MONTH/DAY/YEAR EXPLORATION COMPLETED.
EL.511±	ELEVATION OF GROUND SURFACE AT TIME OF EXPLORATION.

MAXIMUM ARTESIAN HEAD.
SUBSURFACE WATER LEVEL.
ARTESIAN FLOW ENCOUNTED.
RANGE OF SUBSURFACE WATER LEVEL DURING PERIOD OF OBSERVATION.
BOTTOM OF OBSERVATION WELL.
SM — GROUP LETTER SYMBOL ACCORDING TO UNIFIED SOIL CLASSIFICATION SYSTEM.
NR — NO RECOVERY OR UNSATISFACTORY SOIL SAMPLES RECOVERED.
NS — NOT SAMPLED. (CORE DRILLED, BLASTED AND/OR WASH BORED).
24 — NUMBER OF BLOWS PER FOOT OF PENETRATION USING A 350 POUND HAMMER FALLING FREELY AN AVERAGE DROP OF 18 INCHES TO DRIVE A SAMPLE SPOON WITH BEVELLED AND SHARPENED DRIVE SHOE.
* — BLOW COUNT NOT RECORDED OR NOT CONSIDERED REPRESENTATIVE.
COBBLE OR BOULDER. (CORE DRILLED)
COARSE GRAVEL OR NESTED COBBLES AND BOULDERS. (CORE DRILLED AND/OR BLASTED AND CHOPPED).
EL. 482.1 ELEVATION OF BEDROCK SURFACE.
ROCK CORE RECOVERY 0-25 %.
ROCK CORE RECOVERY 25-50%.
ROCK CORE RECOVERY 50-75 %.
ROCK CORE RECOVERY 75-90%.
ROCK CORE RECOVERY 90-100 %.
EL. 461.4 ELEVATION AT BOTTOM OF EXPLORATION.

* NPT
50psi
10 5 0 LOSS IN gpm — 100 0 R.Q.D. — JOINT STRUCTURE
"C" "B" "A"

NOTES:

A. DIP AND FREQUENCY OF JOINTS. "S" INDICATES SLICKENSIDE SURFACE ON NATURAL JOINT SURFACE.
B. ROCK QUALITY DESIGNATION - PERCENT OF CORE PER RUN WITH PIECES IN EXCESS OF 4 INCHES IN LENGTH.
C. N P T - NO PRESSURE TEST PERFORMED. ASTERISK - DENOTES SECTION COULD NOT BE SEALED FOR TESTING. 50 p s i - CONSTANTLY MAINTAINED PRESSURE FOR 1 TO 5 MINUTES. VOLUME LOSS IN GALLONS PER MINUTE UNDER CONSTANT PRESSURE. TESTED CONTINUOUSLY IN 5-FT. SECTIONS. SCALE EXPANDED FROM 0 TO 1 G P M FOR CLARIFICATION OF LOW PRESSURE LOSSES.

SAFETY PAYS

GRAPHIC SCALES

REVISION	DATE	DESCRIPTION	BY

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

WATER RESOURCES DEVELOPMENT PROJECT
PARK RIVER LOCAL PROTECTION
AUXILIARY CONDUIT
INTAKE AND OUTLET SHAFTS
GEOLOGIC - LOG PROFILE
HARTFORD CONNECTICUT

DES. BY | DR. BY | CK. BY | SUBMITTED | CHIEF, SECTION | APPROVAL RECOMMENDED | CHIEF, TECH. ENG. BRANCH | REVIEWED | PROJECT ENGINEER | APPROVAL RECOMMENDED | CHIEF, BRANCH | APPROVED | DATE | CHIEF, ENGINEERING DIVISION | SCALE | SPEC. NO. | DRAWING NUMBER | SHEET

PLATE. 8A-6

APPENDIX A

ROCK LOGS AND TEST DATA

CONTENTS

TABLE

FIGURES

ROCK CORE LOGS

LABORATORY TEST PROCEDURES

Due to lack of well defined laboratory procedures, the following tests are briefly described;

1. Sliding Friction on natural bedding joints were tested in a direct shear device using a cylindrical shear box adapted to accommodate "NX" diameter rock core. All sliding friction tests were conducted at a constant strain range of 0.008 to 0.012 in/min. and at normal stresses of 5, 10 and 15 T.S.F., and all open joint surfaces were in an immersed condition. For laboratory notes and typical test data, see Appendix A, Figure A-2.

2. Multi-Stage Triaxial Tests on saturated undrained specimens were conducted in accordance with CRD-C 147-68 and as outlined in Appendix "C" of MRD First Interim Report, dated July 1966, titled, Strength Parameters of Selected Intermediate Quality Rocks. All cores were surface prepared to within required tolerances using Concinnati centerless and Pope standard surface grinding machines. Samples were tested at constant rates of strain and at maximum confining pressures equal to 5, 10 and 20% of their typical unconfined compressive strengths, total duration of these tests ranging from 2 to 20 minutes. For laboratory notes and test data see Appendix A, Figures A-3 and A-4.

3. Modulus of Elasticity and Poisson's Ratio were computed from results of controlled strain measurements taken during unconfined compression testing data obtained separately on rock core specimens. In this connection, strain

TABLE 1

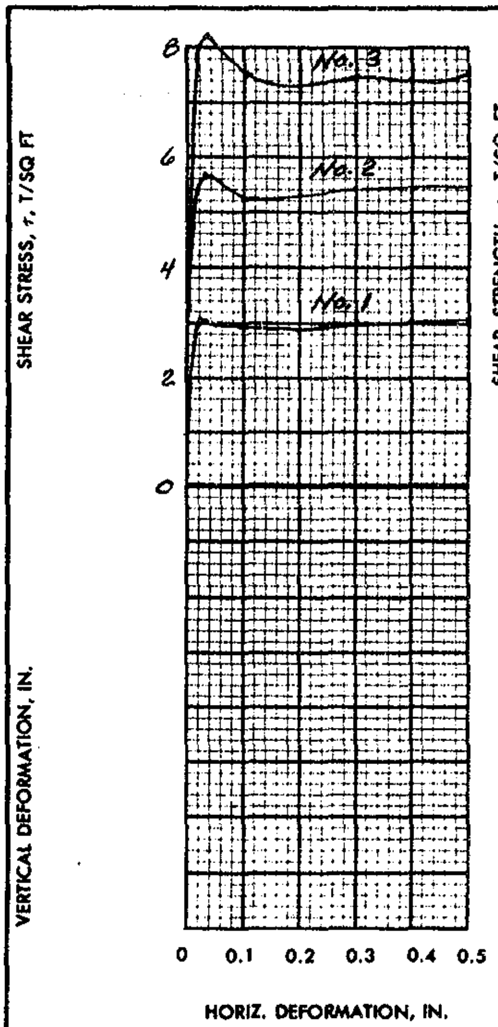
PARK RIVER AUXILIARY CONDUIT
DM NO. 8 SHAFTS
TEST DATA SUMMARY

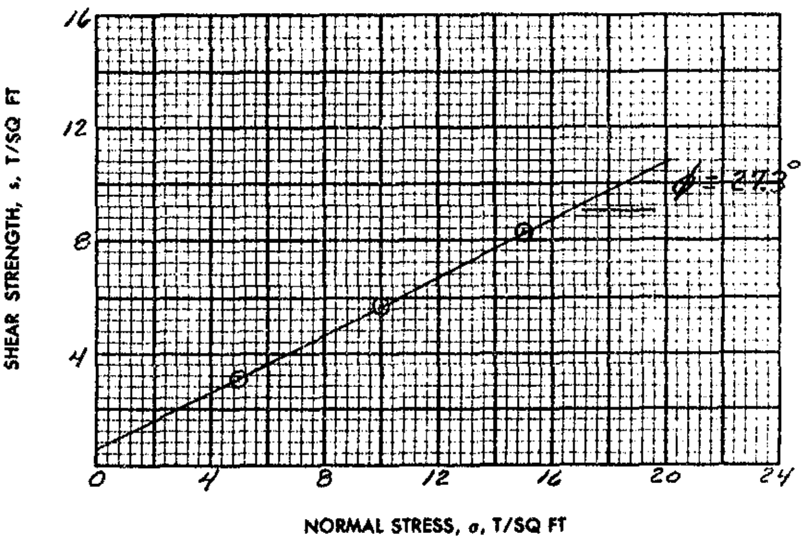
Boring No.	Lab Test No.	Depth (ft)	Apparent Ga	Bulk Dry GM	Bulk (SSD) GMSSD	Absorption %	Unconfined Compressive Stress, psi
FD-25T	1	91.1-92.0	2.73	2.68	2.70	0.8	6,340
FD-25T	2	121.9-123.1	2.73	2.66	2.68	0.9	9,740
FD-25T	3	129.6-130.9	2.69	2.64	2.66	0.7	-
FD-25T	4	131.0-132.0**	2.71	2.65	2.67	0.9	-
FD-25T	5	138.7-139.7	2.69	2.62	2.65	0.9	-
FD-25T	6	155.7-157.0	2.73	2.66	2.68	0.9	5,441*
FD-25T	7	168.0-169.1	2.75	2.65	2.68	1.4	3,242
FD-26T	1	99.3-100.3**	-	-	-	-	-

* For Poisson's Ratio and Modulus of Elasticity values see stress-strain curves

** For shear strength results see Multi-stage Triaxial Test Report

TABLE NO. 1





TEST NO.			1	2	3	
INITIAL	WATER CONTENT	w_o	%	%	%	%
	VOID RATIO	e_o				
	SATURATION	S_o	%	%	%	%
	DRY DENSITY, LB/CU FT	γ_d				
VOID RATIO AFTER CONSOLIDATION		e_c				
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}				
FINAL	WATER CONTENT	w_f	%	%	%	%
	VOID RATIO	e_f				
	SATURATION	S_f	75+ %	75+ %	75+ %	%
NORMAL STRESS, T/SQ FT		σ	5.0	10.0	15.0	
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	3.15	5.71	8.26	
ACTUAL TIME TO FAILURE, MIN		t_f	2.4	3.0	4.4	
RATE OF STRAIN, IN./MIN			0.008	0.008	0.008	
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	2.88	5.23	7.39	

SHEAR STRENGTH PARAMETERS

ϕ' = 27.3°

TAN ϕ' = 0.516

c' = 0.55 T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN Rock Core			2.125 IN. ~~SQUARE~~ Diam.	IN. THICK
CLASSIFICATION Sliding Friction on Natural Open Joint				
LL	PL	PI		G_s

REMARKS (1) Same specimen under different normal loads used throughout series.
(2) Cohesion value is "No-Load" strength of rock combination and results from changing roughness of interface between the 2 halves.

PROJECT Park River Auxiliary Conduit	
AREA	
BORING NO. FD-19	SAMPLE NO.
DEPTH 36.7' - 37.1'	DATE July 1976

DIRECT SHEAR TEST REPORT

ENG FORM 2092 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO: 1966 OF—214-948

Figure A-1

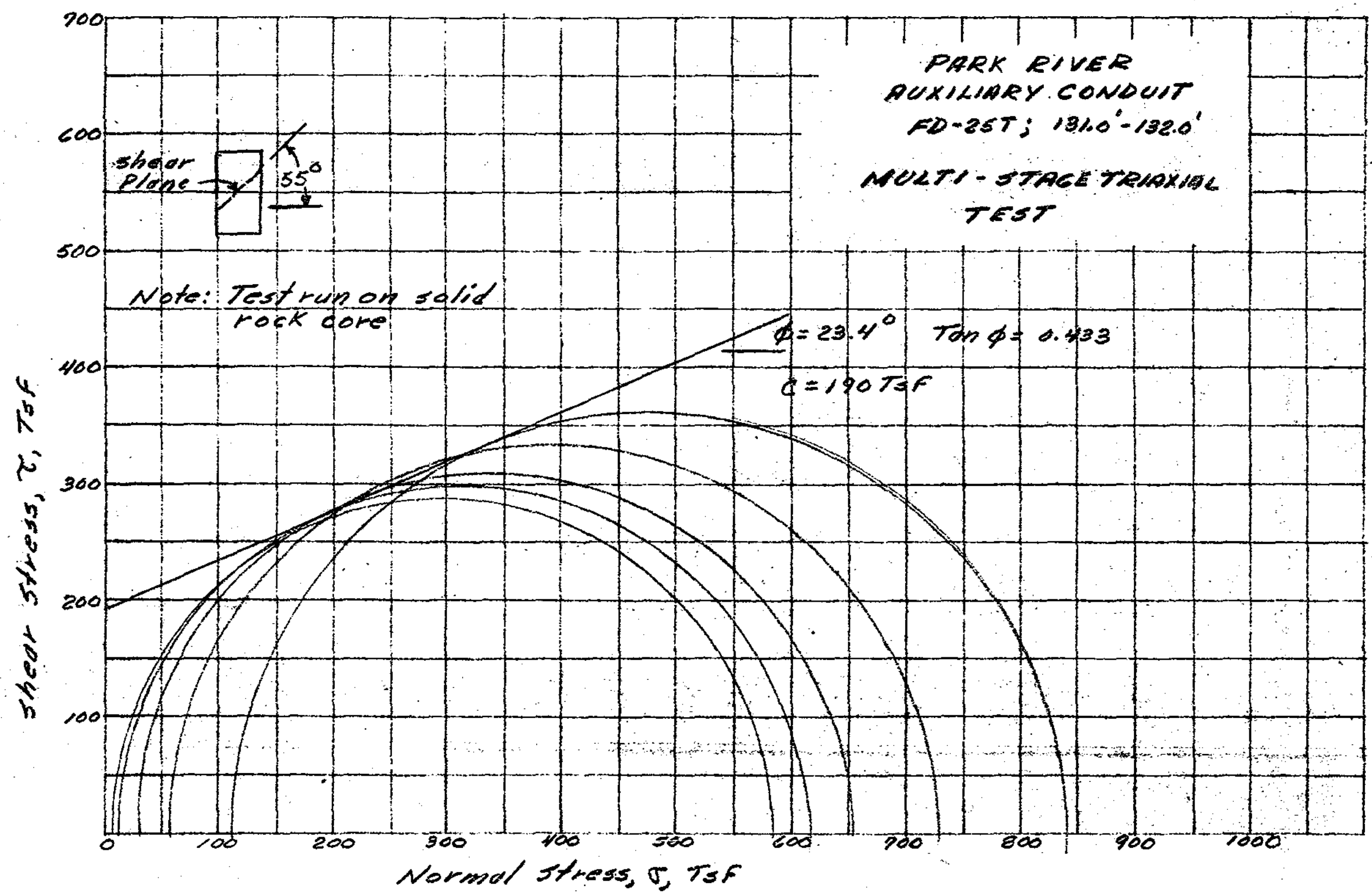

PARK RIVER
AUXILIARY CONDUIT
FD-25T ; 131.0'-132.0'
MULTI-STAGE TRIAXIAL
TEST
shear Plane
55°
Note: Test run on solid rock core
φ = 23.4°   Tan φ = 0.433
C = 190 TSF
Shear Stress, τ, TSF
Normal Stress, σ, TSF
0
100
200
300
400
500
600
700
800
900
1000

Figure A-2

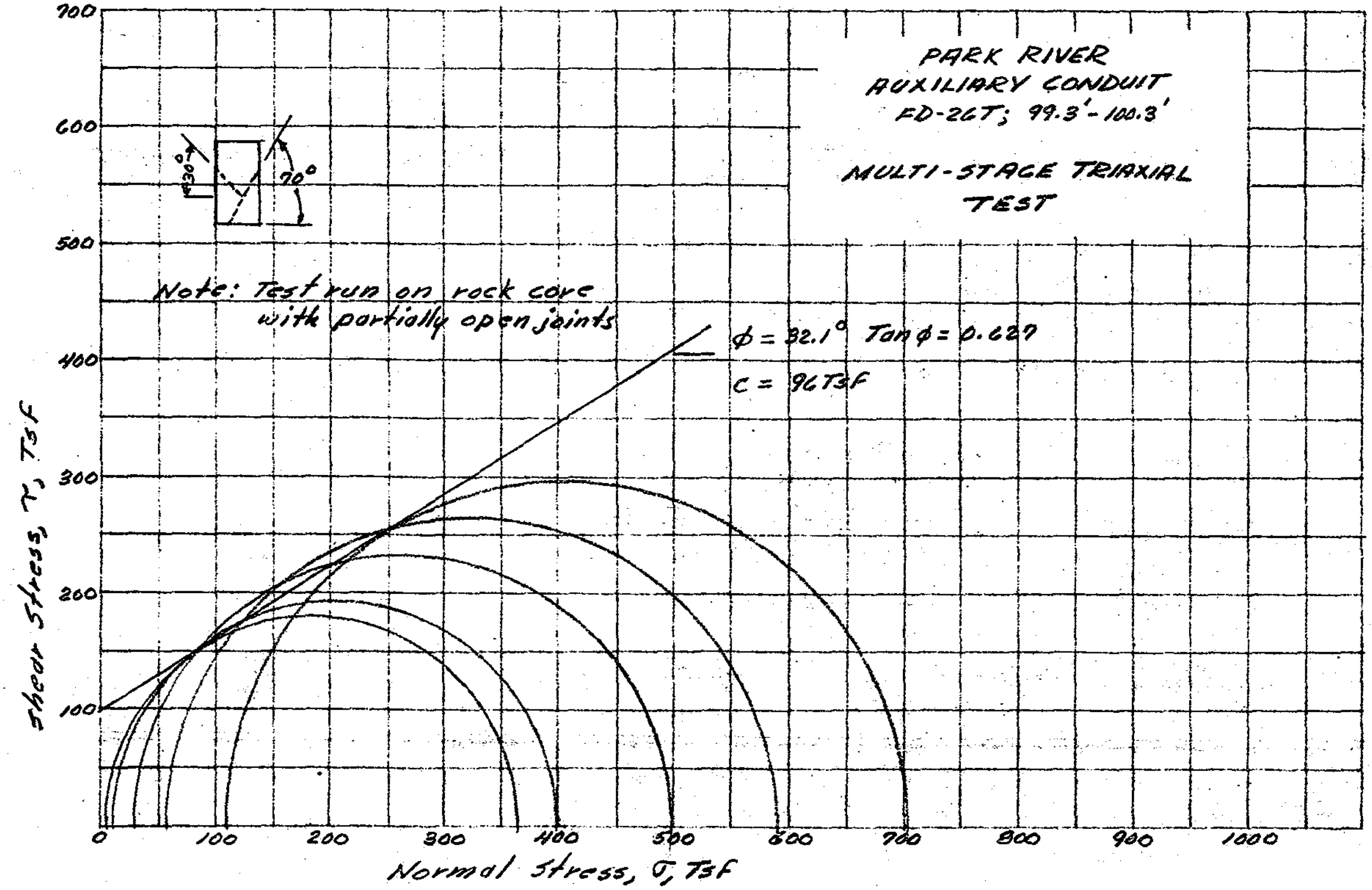
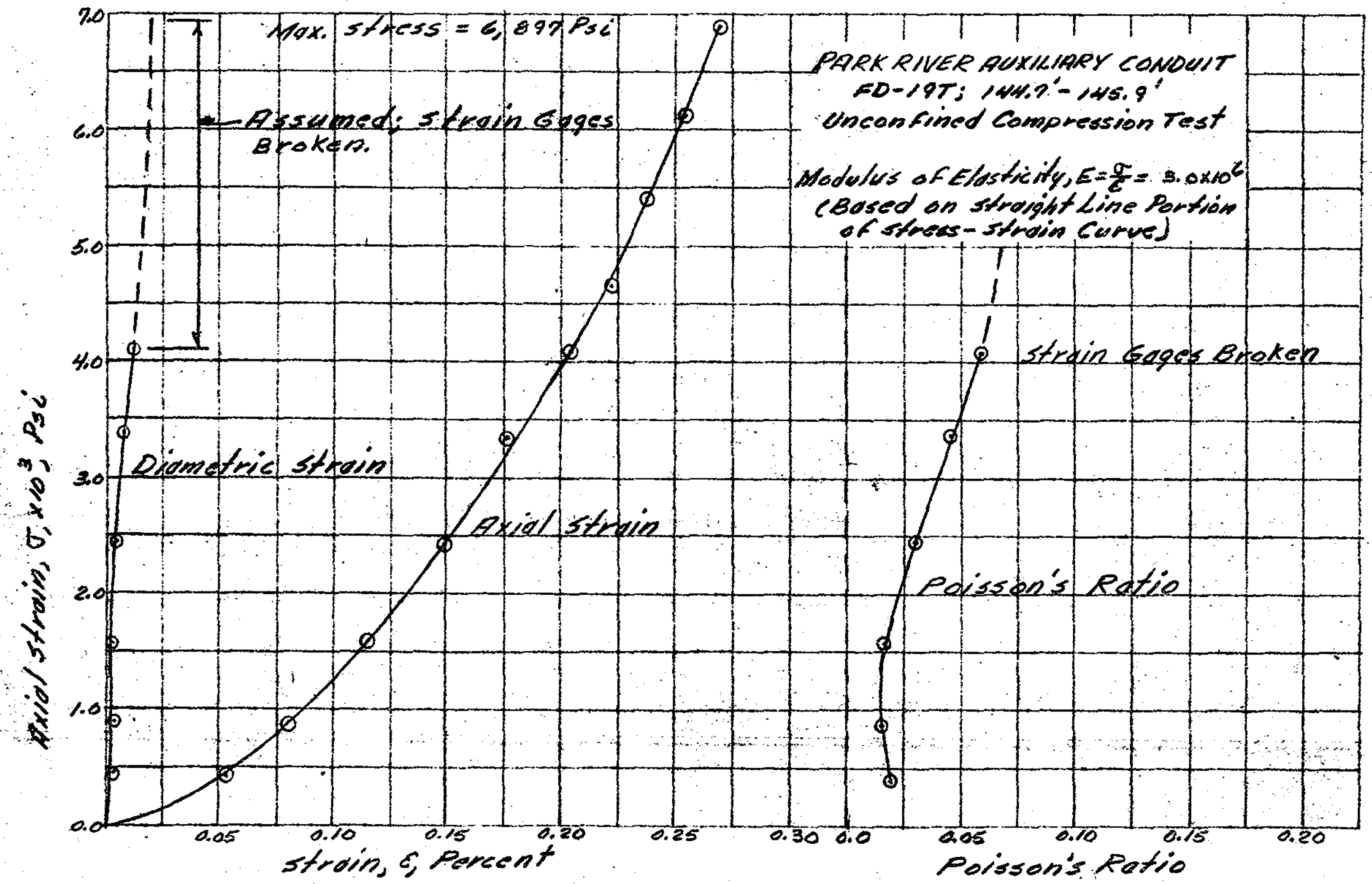

PARK RIVER
AUXILIARY CONDUIT
FD-26T; 99.3'-100.3'
MULTI-STAGE TRIAXIAL TEST
30°
70°
Note: Test run on rock core with partially open joints
φ = 32.1° Tan φ = 0.627
C = 96 TSF
Shear Stress, τ, TSF
Normal Stress, σ, TSF
0
100
200
300
400
500
600
700
800
900
1000

Figure A-3



Max. stress = 6,897 Psi
PARK RIVER AUXILIARY CONDUIT
FD-19T; 144.7' - 145.9'
Unconfined Compression Test
Modulus of Elasticity, E = σ/ε = 3.0x10^6
(Based on straight Line Portion of stress-strain Curve)
Assumed; Strain Gages Broken.
Strain Gages Broken
Diametric Strain
Axial Strain
Poisson's Ratio
Axial Strain, σ, x10^3, Psi
Strain, ε, Percent
Poisson's Ratio
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
0.05
0.10
0.15
0.20
0.25
0.30
0.0
0.05
0.10
0.15
0.20

Figure A-4

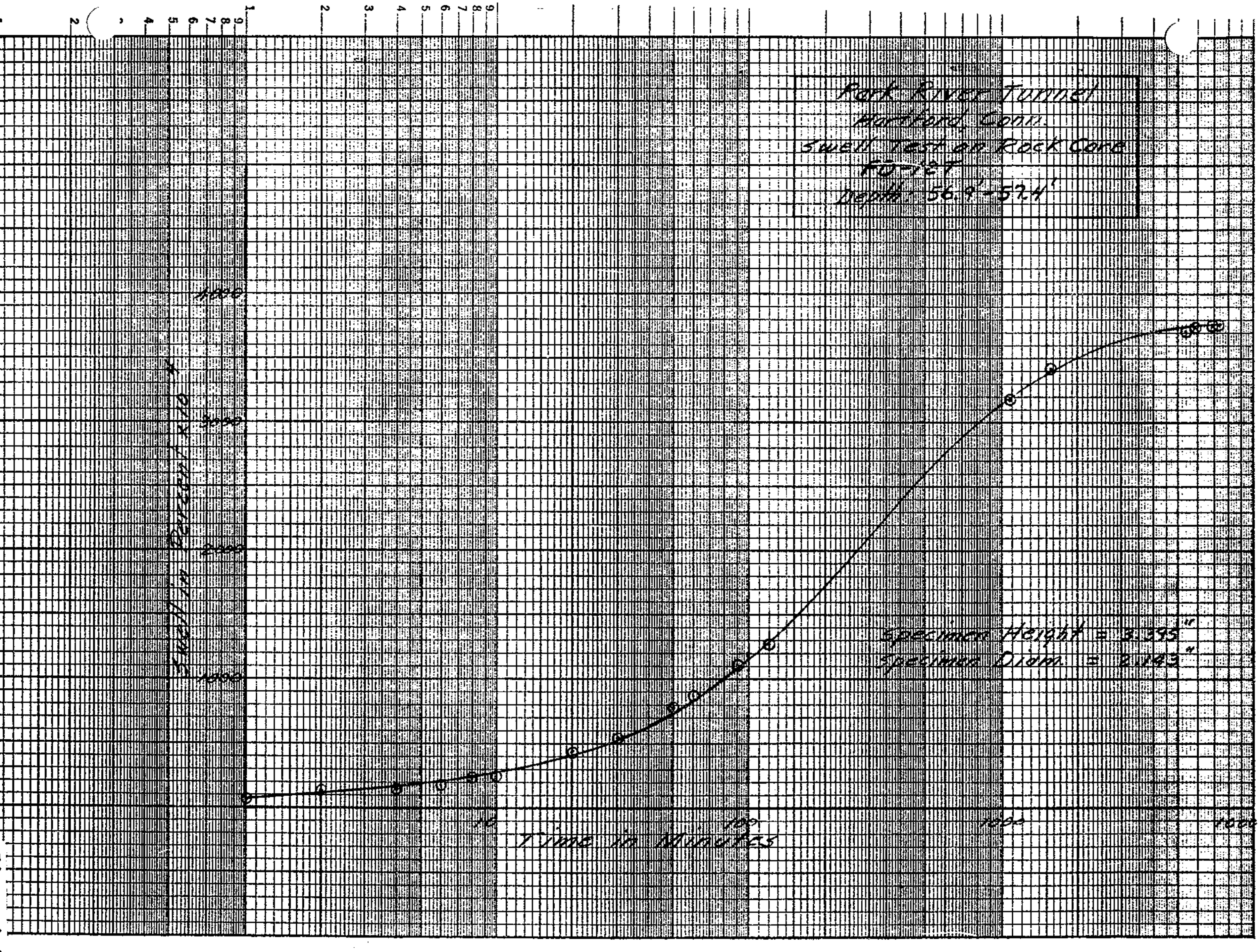

Park River Tunnel
Hartford, Conn.
Swell Test on Rock Core
FD-12T
Depth: 56.9'-57.4'
4000
3000
2000
1000
10
100
1000
Time in Minutes
Specimen Height = 3.555"
Specimen Diam. = 2.143"

Figure A-5

Site:	Boring No.	Page ____ of ____

DEPTH		CORE/SAMPLE			BLOWS PER FT. / CORE REC'VY	SAMPLING AND CORING OPERATIONS					CLASSIFICATION OF MATERIALS
	1"=	NO.	SIZE	DEPTH RANGE							
ELEV			TEST		%	RQD %	PT GPM	JOINTS ∠	CODE	REMARKS	

Laboratory Test Sample No. (See Appendix A Table 1)

Per cent Core Recovery Per Run

ROCK SYMBOL

SHALE

INTERBEDDED SHALE AND SANDSTONE

SANDSTONE

LIMY ZONES

Joint and Dip Angle

Pressure test takes in GPM at 50 p.s.i.

Rock Quality Designation - Per cent of core in pieces in excess of 4 inches in length (per run)

JOINT CODE KEY

3. JOINT ROUGHNESS NUMBER

A. Discontinuous
B. Rough or irregular, undulating
C. Smooth, undulating
D. Slickensided, undulating
E. Rough or irregular, planar
F. Smooth, planar
G. Slickensided, planar

4. JOINT ALTERATION NUMBER

A. Tightly healed, hard, non-softening, impermeable filling
B. Unaltered joint walls, surface staining only
C. Slightly altered joint walls
D. Silty or sandy-clay coatings, small clay fraction (non-softening)
E. Softening or low friction clay mineral coatings (discontinuous coatings, 1-2mm or less in thickness)

LEGEND FOR TYPICAL BORING LOG

Site: Park River Tunnel Hartford Conn

Boring No. FD-25T

Page 1 of 1

DEPTH (1" = 2.0')	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE REC'VY %	RQD %	Pres Test GPM	Joints ∠	Joints #	Joints Remarks	Symbol
87.0		TEST					10	3c 4B		
				90						
89.0				100						
					70		10 / 20	3B / 3c 4B		
91.0		1		100						
93.0						0.3				
94.0							10	3c 4B		
				100						
96.0							10	3c 4B		
					100					
98.0										
				100		0	15	3c 4B	Calcite Lined	
100.0										
102.0										

CLASSIFICATION OF MATERIALS

SHALE, red-brown, thin bedded to massive. Hard, unweathered to slightly weathered along open joints, Limy, silty with localized zones of Limy sandstone.

Site: Park River Tunnel	Boring No. FD 25T	Page 2 of 7

DEPTH (1" = 2.0')	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE RECV'Y %	RQD %	Pres Test GPM	Joints ∠	Joints #	Remarks	Symbol	CLASSIFICATION OF MATERIALS
102.0		TEST				0					
104.0				100			15	3C 4B			
106.0					96		50°	3D			SHALE, red-brown (as above)
108.0				100		.02	15	3C 4B			
110.0							10°	3C 4B			
112.0											
114.0				100	80	0	10°	3C 4B			
116.0				100							
118.0											

FD 25T

Site: Park River Tunnel | Boring No. FD 25T | Page 3 of 7

DEPTH 1" = 2.0	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. CORE REC'VY %	RQD %	Pres Test GPM	Joints ∡	#	Remarks	Symbol	CLASSIFICATION OF MATERIALS
118.0		Test									
				100	80		10°	4 3C			SANDSTONE, light tan, hard & Sound Calcareous.
120.0							10°	4B			
						.08	10°	3B 4B			SHALE, red-brown as above.
122.0	2										
				98							Frequent fresh breaks along bedding dip 125.3' to 129.3'
124.0											
					95						0.3' Core loss 129.3' to 129.6'
126.0											
				98							
128.0						.12					
130.0	3						10°	3C 4B			
	4			93	83		10°	3B 4B			
132.0											
							10°	3B 4B			
134.0											

Site: Park River	Boring No. FD 25T	Page 4 of 7

DEPTH 1"=20'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	%	RQD %	Pres Test GPM	Joints ∠	#	Remarks	Symbol	CLASSIFICATION OF MATERIALS
134.0		TEST									
							50°	3B 4B			
136.0				100	83		10°	3B 4B			Shale, red-brown (as above)
						0					
138.0							10°	3C			
							10°	4B			
140.0		5		100	72		10°	↓			
142.0						0	10°				
							10°	3C 4B			
144.0				100			50°	3B 4B	Strike Joint		Core grinding broken and fractured 145.4' to 147.0'
146.0							10° 10° 10°	3C 4B			
148.0				100	75	0					
150.0											

RM 50A (Test)

Site: Park River	Boring No. FD 25 T	

DEPTH (1" = 2.0')	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE REC'VY %	RQD %	Pres Test GPM	Joints ∠	Joints #	Remarks	Symbol	CLASSIFICATION OF MATERIALS
150		TEST									
				100			10°	3C 4B			
152.0							10° / 55°	3C 4B / 3B 4B			
							1				
154.0					75						
				98			10°	3C 4B			Shale red-brown (as above)
156.0		6				.2					
							10°	3C 4B			
158.0											
				98			10°	3C 4B			
160.0											
					96						
162.0							10°	3C 4B			
						0					
164.0				100							
166.0											

Boring No.

Site: Park River	Boring No. FD-25T	Page 6 of 7

Depth	Core/Sample No.	Size	Depth Range	Core Recv'y %	RQD %	Pres Test GPM	Joints ∡	Joints #	Remarks	Symbol	Classification of Materials
166		TEST									
				80%							
168.0		7							Fresh fractures 166.5 to 167.5		
170.0				98	69	.1					SHALE, red-brown (as above)
							10°	3C 4B			
172.0							10°	3C 4B			
							65°	3B 4B			
174.0				100							
176.0						Back Pressure (see P.T. Log)	10°	3C 4B			
				97				3C 4B			
178.0					88		10°	3C 4B			
180.0				98							
182.0							10°	3C 4B			

Site: Park River

Boring No. FD 25T

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE REC'VY %	RQD %	Pres. Test GPM	Joints ∡	#	Remarks	Symbol	CLASSIFICATION OF MATERIALS
182.0		TEST		98							
184.0						See P.T. Log / Back Pressure	78°	3B 4B			SHALE, red brown (as above)
186.0				100	85						
188.0											
190.0				88							
191.0											Bottom of Exploration

FORM ... Boring No.

Site: PARK RIVER TUNNEL	Boring No. FD - 26T	Page 1 of 9

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE (Test)	DEPTH RANGE	BLOWS PER FT. / CORE REC'VY %	RQD %	Pres Test GPM	Joints ∠	#	Remarks	Symbol	CLASSIFICATION OF MATERIALS
54.2				106 Blows/foot	0	Testing Incomplete			JAR Sample		
56				33	0						Rock fragments Predominantly gravel & boulder size angular & subangular a few fines were recovered 0.2' @ 55.4 but most were washed out 50-100% water loss above rock. Sandstone boulder 58'-60'
58											
60				0	0						
62											
64				100	21						Top of Rock (in situ) 64.2
66				~~86~~ 100	53						SHALE - Red-brown to gray and black, moderately hard, calcareous, thin bedded (bedding dips 20°-30°), localized weathering, scattered joints and slickensides. Contains silty and sandy zones. Shale and silty shale thin bedded and weakly cemented. Shale cores crack when removed from insitu confined state. Cores separate along cracks when wetted. Sandy zones well cemented.
68							30	3c	- Partly Calcite lined		
70				100	37				(see sheet 2)		

Site: PARK RIVER TUNNEL	Boring No. FD-26T	Page 2 of 9

DEPTH 1"=2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	CORE REC'VY %	RQD %	PRES TEST GPM	Joints ∠	#	Remarks	CLASSIFICATION OF MATERIALS
71–72		TEST							80° - Vertical incipient to partially calcite healed 69.0'-73.4	SHALE Red-brown (as above)
72–74							60°	3E 4A	Partial calcite	Frequent partial calcite lined to open fractures, random high-angle to vertical 81.0'-90.6'.
74				96	76		60°			
76						Testing Incomplete	30	3B 4A	- Calcite	Frequent Lt. grey sandy limy streaks 84.0'-90.6' with some cross bedding
78				98	73		75°	3B 4A	- Calcite	0.2' fragmental bedding joint zone @ 85.6'
80							60°	3C 4B		Dip 25°
82				100	0		45°	3C 4A	- calcite	
							80°	3D 4A	- Calcite	
84				96	85		85°		- Parallel open calcite crystal lined	Recovered 4.8' inc. 0.3' from above pull short pull above to 100% & 0.2' core loss in fracture zone @ 85.6'
86							58° 35°	3C 4D		

Site: PARK RIVER TUNNEL

Boring No. FD-26T

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE TEST	DEPTH RANGE	BLOWS PER FT. CORE REC'VY %	RQD %	Pres Test GPM	JOINTS ∠	#	REMARKS	Symbol	CLASSIFICATION OF MATERIALS
											SHALE red-brown (as above)
88											
				100	67			3B 4C	- leached calcite open		
90											90.6' SHALE hard, grey to black. w/ lt. grey limy sandy (25°±) streaks, occas. calcite veinlets.
						Testing Incomplete	25° 60° 25°		- tight calcite lined		
92											
							25° 45°	3C 4A	- Calcite slickensides		
94											
				100	90				3B4B, 3D4A, 3B4A calcite		0.15' thin-bedded to fissile carbonaceous black
96											96.4' Gradational color change
								3A	- incipient only		
							45°				SHALE, red-brown, variably limy & sandy, occas calcite veinlet, hard (as above) hard to very hard sandy, limy cross bedded 97.3'-98.8' 110.8'-115.5'
98								3B 4A	- calcite		
								3C 4A	- Calcite		
				100	79		30°		- 0.1' fissile w slickensides		
100	1										
								3A	- incipient		
102											

Site: Park River Tunnel	Boring No. FD-26T	Page 4 of 9

DEPTH 1"=2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. CORE REC'VY %	RQD %	Pres Test GPM	Joints ∡	#	Remarks	Symbol	CLASSIFICATION OF MATERIALS
		Test						3B 4A	- Calcite		SHALE RED-brown (as above)
104				100	79				slickensides		
106							60	3B 4A	- Calcite		Mod. slaking parts also along partly calcite healed joints 107.0'-110.8' ; 115.0'-120.1'
							25°		slickensides - 3B 4A Calcite		
						Testing Incomplete	25°		- 3A 4A "		
									- 3C 4A "		
108				100	22						
110											
112											gained 0.3' this pull ∴ increased pull above to 100%
114				100	26		45°	3B 4C	open w/ Calcite crystals		
									trace slickensides		
116							45°				
							10	3B 4A	- calcite slickensides, polish		
118							25°		4 Jts 3D 4B		

FD-26T

Site: Park River Tunnel | Boring No. FD-26T | Page 5 of 9

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE REC'VY %	RQD %	Pres Test GPM	Joints ∠	#	Remarks	Symbol	CLASSIFICATION OF MATERIALS
		TEST									
				100	59				Vert. Joint Partially Calcite lined Core is fragmental 117.6'-120.2		Small shale fragments exhibit polished surfaces @ 120.0'
120											
									70° 3C 4a Calcite		
122				100	60	Testing Incomplete			- Slaked 30° Cross beds		Slaked along thin sandy limy + 30° cross beds 121.9'-123.0'
124											Very Sandy - limy 121.9 - 124.4
							30°	3C 4C	Scattered Calcite healed Joints 125.3' - 126.6'		
126											Gain of 0.7' this pull to readjust pull above to 100%
				100	93						
128							30	3B 4B	Thin calcite lined irreg-vert Jt. 128.0'-130.0' - 20°-40°		
130								3B 4B			0.2 Impure micaceous Calcite @ 126.8' & 131.7'
							30				
							20	3C 4B			
132				96	81		25°	3C 4A	- Calcite		
134											

Site: PARK RIVER TUNNEL	Boring No. FD-26T	Page 6 of 9

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE REC'VY	SAMPLING AND CORING OPERATIONS					Symbols	CLASSIFICATION OF MATERIALS
		TEST		%	RQD %	Pres Test GPM	JOINTS ∡	#	REMARKS		
							70°		- 3B 4A Calcite		Parts along random hairline calcite filled fractures 136.0'-139.0'
							60°		3C 4B		
							35°				
136							10°		1/8" calcite		Gain of 0.5' this pull applied to recovery of pull above.
138				100	71						SHALE Red-brown (as above)
140						Testing Incomplete					
142				100	79		30°				SANDY-Limy cross beds, Lt grey hard - very hard w/ calcite veinlets near normal to ±30° cross beds 140.5'-142.0
144									-Bedding Jts 30° 3C 4B (142.7-145.0') Vert. Joint partially calcite healed.		Mechanically Broken by going over 144.3-144.8'.
146							20°				
				100	62			3C 4B			Irregular partings along discontinuous calcite veinlets and along bedding planes which average 0.1' thick. 147.6'-150.2
148							20°				
150											

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. / CORE REC'VY %	RQD %	Pres Test GPM	Joints ∠	Joints #	Remarks	Symbol	CLASSIFICATION OF MATERIALS
		TEST					30°, 20°, 30°	3C 4B			SHALE Red-brown (as above)
152				100	63		30				Increasingly sandy - limy, changing from brown to grey color w/high-angle discontinuous calcite filled fractures and cross beds 151.0' - 154.5'
							70°	3D 4B			
154								3C 4D	(2) 90° joints solutioned		154.5
								3G 4C	- 0.2' fissile zone		SHALE, dk. grey to black w/ numerous thin calcite filled fractures, mod hard Tends to slake.
156						Testing Incomplete		3G 4B	- Bedding planes		
				100	54		20		Fragmental joint zones 0.1' @ 156.0', 0.2' @ 158.1', 0.2' @ 158.7, 0.4' @ 159.2		Very sandy - limy w/ cross beds 156.0' - 156.9'.
158							60°	3D 4D			
160							20°, 45°		fract. zone w/ thin clay coat		
				96	60						
162											
164											
									trace slickensides		
166											

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	CORE REC'VY %	RQD %	Press Test GPM	Joints ∡	#	Remarks	Sym.	CLASSIFICATION OF MATERIALS
		TEST									
168				96	58		20°	3A 4B	Very small fragments - Some exhibit polished "Slickensides" 166.5'-167.0		SHALE, grey (as above) 0.2' probable core loss zone in highly fragmental core dist. 166.6'-167.4.
170 / 172				100	40	Testing Incomplete		3B 4C	0.05' Brecciated 45° Joint @ 170.4' Numerous Random Fractures Slickensided 171.0'-171.6'		Core highly broken about 0.3 gain 168.4-173.0 & 0.2' gain 173.0'-177.7 to adjust prior pulls.
172.5							55°, 30°, 2?	3C 4C	- open w/ calcite; slickensided		Highly sandy & limy, cross bedded w/ leached random calcite veinlets, fragmental 172.2'-174.2'
174 / 176 / 178				100	39				incipient slickensides		SHALE redish-brown, variably silty & limy, hard w/ random white calcite veinlets ~~tends~~ tends to slake along bedding and part along calcite lined discontinuous fracture planes Dip varies 10° to 40°
180 / 182				100	92		10°, 15°, 25°, 30	3B 4B	Bedding Joints w/ Some calcite lining.		

Site: PARK RIVER TUNNEL

Boring No. FD-26T

DEPTH 1" = 2'	CORE/SAMPLE NO.	SIZE	DEPTH RANGE	BLOWS PER FT. CORE REC'VY %	RQD %	Pres Test GPM	Joints ∡	#	Remarks	Symbol	CLASSIFICATION OF MATERIALS
		TEST						3B			
							26°	4B			3.0' Core gain this pull to adjust prior pull to 100%
				100	72						
184											SHALE Red-brown as above
							60°	3C 4A	- Calcite		
186											
				100	28	Testing Incomplete	20°	3B			
							60°	4C			
188									3A 4A Calcite		
							15°	3A 4C	- 70° - partly calcite healed		
							25°	3D			
190							60	4B	3E 4B Slickensides		
192				82	72		25°	3E 4B			0.9' Core loss this pull & 0.6' Core loss in final pull location is not apparent - core likely left in hole - core catcher not holding.
194							10°	3E 4B			
							15°				
196							10				
				68	37		60				
							30				
							60°		slickensides		
198											

← Bottom of Exploration

APPENDIX B

FIELD LOGS AND SOILS DATA

CONTENTS

FIGURES (FD-13U)

PLATES

FIELD LOGS